Assessment of the Design of Stormwater Ponds for Flow Attenuation and Water Quality Treatment

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Abstract

In order to reduce the impact of flooding and water quality degradation in urban areas sustainable urban drainage systems (SUDS) are increasingly being implemented throughout the UK. The thesis is concerned with one such type of system, namely retention ponds. With an absence of continuous long-term monitoring data to demonstrate how well these ponds perform in practice, a mathematical model was developed to investigate their flow attenuation and water quality enhancement characteristics. Simulations obtained with the model aimed to quantify how well ponds, designed using current UK guidance, are likely to perform now and under climate change scenarios. Furthermore, the model was used to study the effect of innovations in pond design. Initial modelling concerned ideal, generic ponds, with the knowledge gained being used to guide a case study on Linburn Pond in Scotland. Results show that the volume of temporary storage and the design of the outlet device are both of critical importance in meeting both flow attenuation and water quality enhancement targets. Furthermore, results also indicate the importance of dilution in achieving water quality targets. Simulations show that not only should a large permanent pool be provided but that water quality performance improves significantly when this volume is provided using larger surface areas as opposed to by deeper permanent pools. The assimilation of the knowledge gained in the study has enabled a set of improvements to current retention pond design to be proposed.

Dedication

This work is dedicated to two gentlemen who unknowingly, greatly influenced the writing of the thesis and, who would be very proud of this achievement. Firstly, it is dedicated to the memory of my Father, Thomas Morgan whose love of the natural world filled me with wonder from an early age. It is also dedicated to my Uncle, James McCusker for his constant love, support and encouragement throughout my education.

Acknowledgements

There are a number of people I would like to thank, without whose help, support and advice, the thesis could not have been written. Firstly I would like to extend my thanks to my supervisors, Dr. Steve Wallis, Dr. Rebecca Lunn and Dr. Kate Heal for their input, advice, and their tireless efforts in correcting the early drafts of this work. I am grateful to Scottish Water and Heriot-Watt University for their joint funding of my studies. I would also like to thank Dr. Chris Jeffries and Dr. Adolf Spitzer of the Urban Water Technology Centre at Abertay University and Neil McLean from SEPA for providing me with the much needed data for retention ponds in Scotland. Finally I would like to thank all of my family and friends for their encouragement throughout the project. In particular, I would like to thank my mother, Josephine Morgan for always being there for me and my partner Nicholas Clark for his endless support, advice and encouragement – and most of all, for believing in me. I would also like to thank Helen Crow and Heather Haynes, my good friends and colleagues, for their constant support and cheerfulness which made even the most difficult of days fun.

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1 Introduction

Sustainable Urban Drainage Systems (SUDS) are relatively new concepts in the UK. They are implemented as part of a stormwater drainage system with the aim of minimising the impact of urbanisation on the water environment. Retention ponds are one such type of system developed to perform the dual purpose of attenuating high storm flows and reducing the adverse water quality impacts that occur with urban development. Since these systems have only been introduced and implemented in the UK in the last ten years, their effectiveness in reaching their flow and water quality targets has not yet been quantified. Retention ponds have been implemented in other countries for many years, however, they have been shown to be highly site specific and the transfer of data for the purposes of modelling has not been successful. This research models the flow and water quality performance of retention ponds.

1.1 Pond Design for Optimum Performance

A pond's flow attenuation and water quality performance is entirely dependant upon its design for the range of storm conditions it is expected to encounter over its operating life. The standard design manuals used in the UK provide a number of methods for the adequate design of storm water retention ponds. Theoretically, ponds designed using these manuals should meet both flow and water quality objectives. Obstacles that might prevent this include exceedence of expected storm conditions, poor maintenance, or financial restrictions that prevent the optimum pond design being implemented.

Flow attenuation in ponds is achieved by providing a temporary area of storage for incoming stormwater. Due to the dual function of retention ponds, they must be sized in order to provide a permanent pool as well as a temporary storage volume. In the UK ponds are sized based on attenuation of a given design storm. Such an approach assumes that the frequency of occurrence of the design storm will remain constant over the pond's operating life. However recent General Circulation Model predictions suggest that changes in climate may affect the severity of storms in the UK with probable increases in storm magnitude and frequency. Conditions that exceed the design parameters of the pond may severely compromise pond flow attenuation performance.

In the past retention ponds were built with the sole aim of reducing high flows that might pose the risk of catchment flooding. However more recently, with concerns over the impact of high pollutant loads on receiving waters, ponds are now designed with water quality as the primary focus. This water quality function is achieved by providing a quiescent body of water, known as the permanent pool, to enhance particle settling. The permanent pool is also referred to as the treatment volume due to its role in water quality improvement. It is a low energy environment that acts as sediment sink for particulate-bound pollutants and is *the* characteristic feature of retention ponds.

In theory these two basic functions of flow and sediment attenuation in retention ponds play competing roles: for stormwater flow abatement large storage capacities are required, and ponds are required to drain quickly to provide adequate storage for the subsequent storm; however, large treatment volumes (i.e. ponds with large permanent pools) and quiescent conditions are desirable for sediment settling. Furthermore as the aim of water quality treatment is achieved, the accumulation of sediment on the base of the pond will negatively affect a pond's ability to attenuate flows by reducing the available storage volume. This complex operating cycle will manifest with time and will make it increasingly more difficult for the pond to meet both flow and water quality objectives, particularly if changes in hydrologic regime (such as those induced by climate change) result in conditions that exceed the design parameters.

The investigations reported in this thesis utilise a numerical model to predict flow attenuation and water quality performance of retention ponds and detention basins under both single and multiple event scenarios. The model simulates hypothetical generic ponds to quantify flow and water quality performance and investigates the optimum pond design for meeting both flow and water quality targets.

1.2 Aims and Objectives

The investigations outlined here are designed with the general aim of identifying the major influences on pond flow and water quality performance and determining how pond design can be best achieved to optimise these. Furthermore, they aim to identify the major obstacles that prevent ponds from achieving their future flow and water quality targets.

To achieve this aim, specific objectives include:

1. Quantifying the flow attenuation performance of retention ponds
2. Investigating pond design for optimum pond flow attenuation performance
3. Assessing how performance might change under changing climatic conditions.
4. Quantifying the water quality performance of retention ponds and investigating the potential water quality performance of detention basins
5. Investigating pond design for optimum water quality performance
6. Assessing how ponds can be designed to achieve both flow and water quality targets and identifying the obstacles that might prevent it
7. Proposing improvements to design guidelines

These objectives are met in the following chapters following a literature review presented in chapter 2; Chapter 3 describes how flow is modelled and presents the major influences on pond flow attenuation. Chapter 4 employs data from a functioning retention pond, Linburn Pond, and assesses how it could be improved under present and future climate change scenarios. Chapter 5 describes how water quality is simulated and presents initial simulations that determine the key influences on sediment retention in ponds. Chapter 6 investigates different pond configurations to determine the optimum design for pond water quality improvement. Chapter 7 provides a brief summary of results as well as conclusions and recommendations for future work. For completeness, Appendix E contains 4 published papers that are based directly on the work described in the thesis.

2 Literature Review

Following the introduction of the European Union Water Framework Directive, which has been transposed into Scottish law under the Water Environment and Water Services Act (2003), all new urban developments in Scotland are required (with few exceptions) to employ sustainable urban drainage systems (SUDS) as a means of protecting water quality and reducing flood risk (SEPA, 2003). However SUDS systems are relatively new engineering solutions in the UK, hence very little is known about their whole-life performance in temperate climates. Lack of technical guidance and concerns regarding the ability of SUDS to attenuate large flood events and improve water quality throughout their design life may be hindering the uptake of SUDS in the UK [*The Wildlife Trust*, 2000].

Due to the international application of sustainable solutions in stormwater management, the terminology varies widely for different treatment devices in the literature. For the purpose of clarity in this thesis, reference will be made to retention ponds (which are sometimes referred to as wet ponds or detention ponds in other work), detention basins (sometimes known as dry detention ponds or basins) and when referring to both collectively, the terms stormwater management basins or SUDS basins will be used.

This chapter discusses the effects of urban development on catchment hydrology, in particular, the detrimental effects to catchment flows and water quality, and introduces the role of Sustainable Urban Drainage Systems in mitigating these impacts. Also reviewed is the current practice for the design of SUDS basins both internationally, and here in the UK.

2.1 Effects of Urbanisation on Hydrological Response

In an undeveloped catchment, water is distributed by a number of processes that can be described by the hydrological cycle (Figure 2.1). In temperate climates, the natural catchment typically has a high permeability, and precipitation falling on the catchment surface may infiltrate through the soil (where it may contribute to groundwater), may move over the surface (as runoff) or may evaporate (or be transpired by vegetation) back into the atmosphere. Water may also be stored temporarily in ponds and lakes or as snow.

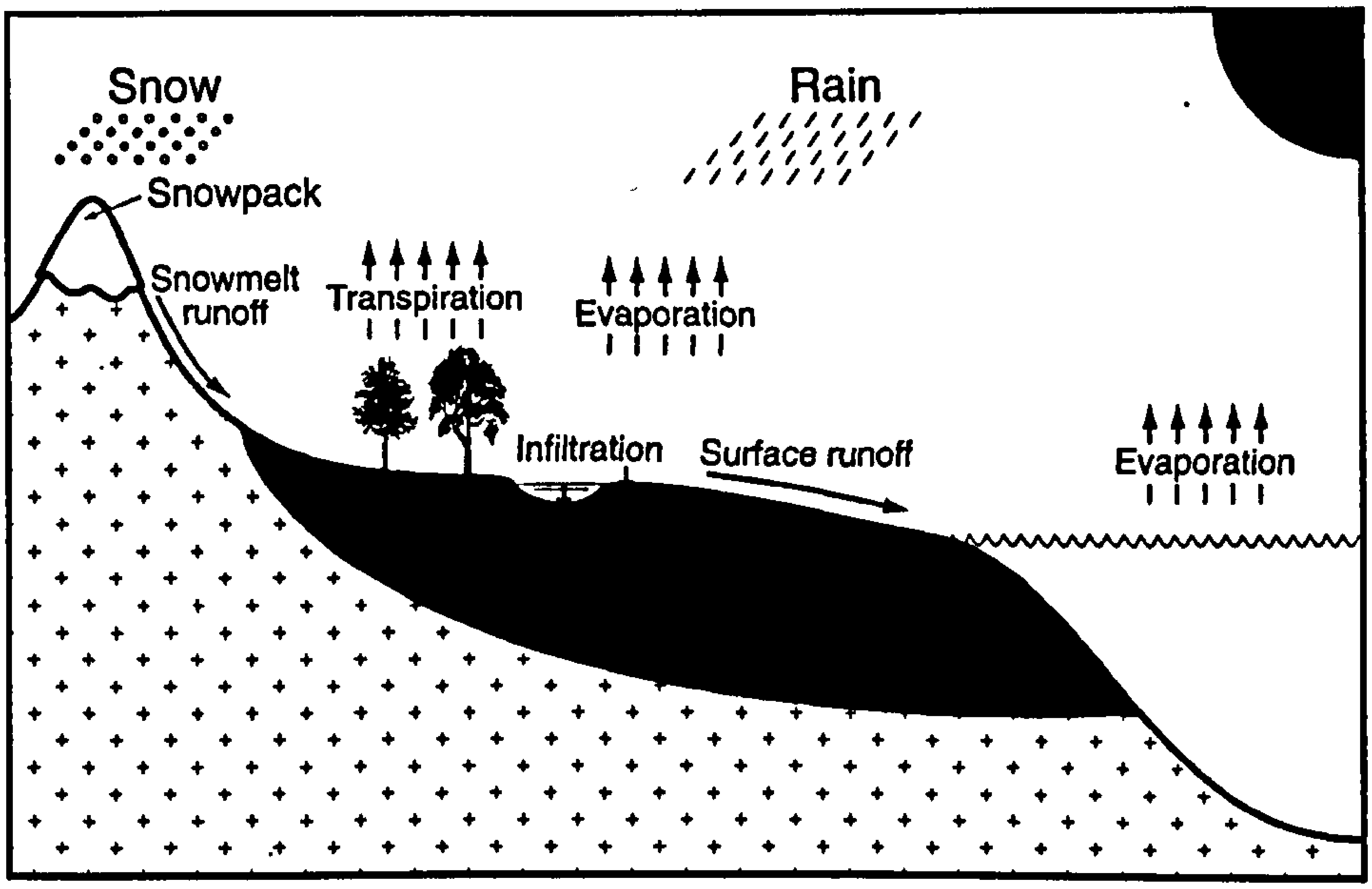


Figure 2.1: The Natural Hydrological Cycle, Source: [*Hornberger et al.*, 1998].

Urbanisation is a process that has multifaceted effects on the hydrological response of a catchment. In a developing urban catchment, vegetation is removed and the area of impermeable surfaces increases with building works. This, in combination with accompanying changes in topography, has significant impacts on the quantity and quality of stormwater runoff [*Sheaffer*, 1982].

2.1.1 Effects on Flow

Both laboratory and field experiments have shown that increasing the impervious area in a catchment adversely affects the volume and timing of surface runoff. As the land surface is developed for urban use, new structures, foundations and roadways increase the impervious area in the catchment, whilst available areas of natural storage, such as soils, surface depressions and ponds are replaced and the water storage capacity in the catchment diminishes considerably. As impervious area in the catchment increases towards 100%, the amount of vegetation, natural surface and infiltration capacity approaches zero [*Lazaro*, 1979]. Recent research by the [*Stormwater Industry Association*, 2003] has shown that 90% of stormwater runoff can be stored via infiltration in a pre-developed catchment as opposed to only 10% in densely urbanised areas. Further, since storage capacity in the urban catchment is reduced by urbanisation, saturation and consequent surface runoff occur much more rapidly, shortening catchment response time to rainfall. In an undeveloped catchment infiltration capacity must be reached before overland flow occurs, whereas in a highly urbanised catchment, impermeable surfaces shed runoff almost immediately, with virtually no losses to the ground surface. This decrease in runoff response time is compounded by highly developed drainage networks, which transport large flow volumes quickly to the nearest watercourse, artificially shortening the time taken for precipitation to reach the river.

The urbanisation of a catchment and the resultant increase in impervious surface cover and development of highly efficient drainage systems has a marked effect on water volume and flow rates in a catchment. Local flooding problems are exacerbated by the consequent magnification in peak flows by 2-4 times those of undeveloped catchments. Studies have shown that unit hydrograph peak flows may triple, whilst rise time is reduced by a factor of three during the course of urbanisation [*Jones*, 1997; *Lazaro*, 1979; *Newson*, 1997; *Sheaffer*, 1982]. There is also a higher frequency of peak flows in rivers associated with the increase in impermeable surface area which may lead to increased channel and bank erosion and the destruction of riverine habitat [*Campbell*, 2004]. Figure 2.2 illustrates the typical changes in hydrological regime caused by urbanisation, which is characterised by magnified peak flows and reduced time to peak for the same rainfall event [*Lazaro*, 1979]. Subsurface hydrology is likewise affected. The increased impermeable nature of the ground surface reduces ground water recharge resulting in a reduction in groundwater levels and baseflows. These problems may be further intensified by high surface water and groundwater demand in developed urban areas [*Campbell*, 2004].

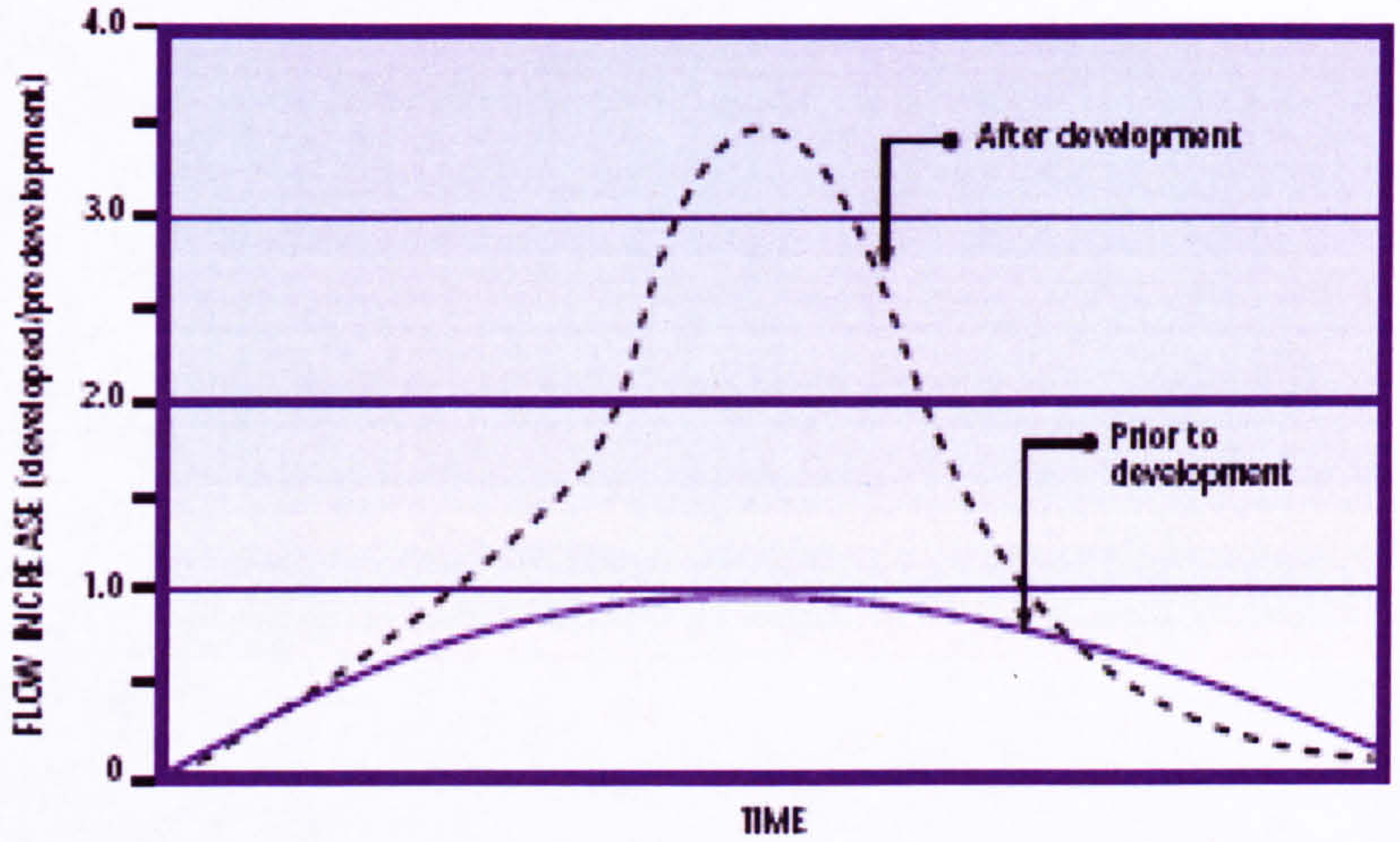


Figure 2.2: Comparison of pre-development and post-development hydrographs, source: [*SUDS Working Party*, 2001].

2.1.2 Effects of Urbanisation on Water Quality

Urban stormwater runoff has contributed to a significant decline in the quality of UK surface waters. It is considered to be diffuse pollution since the nature of the polluting load, and the concentrations associated with it, vary depending on its origin [*Sansalone and Buchberger*, 1997b; *Sansalone et al.*, 1998; *Yeh and Labadie*, 1997]. The contaminant load from urban runoff can be significantly higher than that of secondary treated domestic sewage and is thought to be responsible for 20% of Scotland's rivers being classified as having 'poor water quality'[*Deletic et al.*, 1997]. Table 2.1 presents some of the common constituents of urban stormwater runoff.

Table 2.1:Typical stormwater contaminants, their likely sources and effects, adapted from [*German*, 2003]

CONTAMINANT	SOURCE	EFFECT
Heavy Metals	Building corrosion, fuel combustion, automobile corrosion and industrial materials	Bio-accumulative and may pose a threat to human health in concentrated doses
Organic Pollutants	Pesticides, solvents, oil and grease, combustion of wood and coal	Persistent in the environment and may pose a threat to human health since they are both mutagenic and carcinogenic. Also impacts aquatic ecosystems.
Nutrients	Mainly from atmospheric fallout (nitrogen and phosphorous), and animal droppings (phosphorus)	Nitrogen can be potentially hazardous to human health due to elevated concentrations in drinking water and both nitrogen and phosphorus can cause nutrient enrichment (eutrophication) in standing water.
Pathogenic Micro-organisms (bacteria, viruses, oocytes)	Associated with poorly connected sewers and runoff from pet faeces	May be harmful to human health and a threat to aquatic species.
Gross Pollutants (Rubber, glass, vegetation, sediment and litter)	Mainly from anthropogenic activities such as construction, vehicle use and breakdown of vehicle components and waste disposal	May act as a vector for pathogens, providing a breeding surface and may also cause clogging in the drainage system.

2.1.3 Sources of Pollutants in Stormwater Runoff

The nature of the polluting load in urban stormwater is highly dependent on its origin [*Eriksson*, 2005], as illustrated by Table 2.1. Urban stormwater may also contain runoff contributions from residential areas (including roofs, guttering and driveways), highways and other urban surfaces, as well as from industrial and commercial sites. In urban areas, stormwater is routed from source to sink in highly developed drainage systems.

In the UK there are two types of traditional urban drainage infrastructure: combined sewer systems, which transport both wastewater and stormwater together, and separate sewers, which transport wastewater separately from stormwater. Although the pollution load carried by separate sewers is much lower than that carried by combined systems, it is still highly contaminated and may be similar in quality to treated sewage [*Novotny*, 1994]. Combined sewers are the most common type of drainage system in the UK, accounting for more than 70% of the sewerage network in terms of length. The combined sewer system was originally designed on the assumption that stormwater runoff was benign in nature, and that foul flows would be diluted by the inflowing rainwater. It is now known that stormwater runoff can be highly contaminated with toxic materials. The combination of wastewater with potentially highly polluted stormwater results in the combined system being associated with the highest contaminant load [*Butler and Davies*, 2000]. Consequently, as well as the aforementioned changes to the timing and volume of flows in the catchment, combined sewer overflows (CSOs) may discharge polluted surface runoff and (often untreated) wastewater directly to the river environment [*Lee*, 2000] resulting in a severe deterioration of the water quality of the receiving water system.

Contributions of roof runoff from residential catchments can contain significant concentrations of pollutants [*Fulcher*, 1994]. A study of pollutant contributions from several types of roofs in Bayreuth, Germany, showed that roof runoff pollution is highly influenced by local sources. The roof material itself contributes to this, particularly if it is made from heavy metals such as zinc. Other influences are air pollution (dry deposition), the precipitation event (intensity and antecedent period), meteorology (wind speed, season) and the pollutants' physiochemical properties, which together, can generate a high degree of variability in the nature of the pollutant load from roof runoff [*Fulcher*, 1994; *Garnaud et al.*, 1999].

Numerous studies have been conducted on highways and their contribution to the deterioration of surface waters. Runoff from urban highways is particularly contaminated since it contains many of the typical urban pollutants, namely sediments, hydrocarbons, heavy metals, phosphorous, nitrogen, oil, grease, pathogens, de-icing salts (in winter), as well as a number of materials associated with automobile use [*Barbosa*, 2001]. Such materials are generated from a range of anthropogenic sources such as the combustion of fuels, wear of vehicles, fluid leakage, pavement degradation, highway maintenance, construction activities, road safety equipment and atmospheric deposition and have resulted in runoff which, according to [*Koelman et al.*, 1999] often exceeds Swedish national water quality standards. Of particular concern are the significant quantities of highly toxic (and in some cases carcinogenic) substances such as polycyclic aromatic hydrocarbons (PAHs) and heavy metals such as copper, zinc and to a lesser extent (since the prohibition of leaded fuels) lead [*Koelman et al.*, 1999; *Sansalone and Buchberger*, 1997a]. A further concern is the introduction of a range of metals such as platinum, rhodium, manganese and nickel into urban surface runoff from catalytic converters and fuel additives. The potential for these metals to take part in redox reactions (and thus produce free radicals) makes them a significant threat not only to water quality, but to human health and aquatic life [*Ellis*, 2000; *Fulcher*, 1994; *Hares*, 1999; *McNeil and Olley*, 1998].

It is known that urban stormwater contaminants are highly variable in composition and concentration and come from widely varied sources; however the majority of stormwater contaminants are taken from source to sink by a common vector - sediment. Sediment is the most influential non-point source pollutant, with regard to its mass, [*Deletic and Maksimovic*, 1998b]. To a large extent sediments provide the binding surface to which pollutants, heavy metals and hydrocarbons adhere, and once in the water environment, can persist for long periods of time [*Delleur*, 2001]. It is sediments that determine the fate of the many toxic and bio-cumulative materials that are washed into urban receiving waters [*Ellis and Hvitved-Jacobsen*, 1996]. This is particularly true of the finer sediment fraction (<63μm diameter), which strongly adsorbs pollutants such as heavy metals and pesticides. Sediments are therefore both a sink for, and a potential source of, pollutants [*Ellis and Hvitved-Jacobsen*, 1996].

It is well known that urban activities (such as construction, traffic use, accumulation of litter etc.) produce a range of toxic and persistent contaminants for attachment to solid matter; however, this problem is exacerbated by urban activities which increase the amount of sediment available for contaminant transport. Studies have shown that urban development can increase sediment yield in a catchment by 26 times [*Ostry*, 1982].

The mechanisms that govern the detachment and transport of sediments from the land surface into the receiving water are extremely complex. For the purposes of modelling, the process is often divided into two separate processes; build-up and wash-off [*Deletic*, 2000], as discussed below.

Build-Up

The accumulation of sediment particles (and their adsorbed pollutants) on catchment surfaces is known as the build-up process. It begins immediately after a rainfall event has ended as sediment particles from a variety of sources begin to accumulate on the urban surface during the dry antecedent period. Build-up occurs by three mechanisms: the settling of particles from the atmosphere, the accumulation of particles from local sources and the re-distribution of particles by wind and traffic [*Greenway*, 2003]. The rate of build up is thought to be most rapid during the first few days after a rainfall event and decreases subsequently [*Sartor and Boyd*, 1972]. The highest pollution loads in runoff are likely to be associated with intense rainfall events that have long dry periods between them, since this produces a pattern of high pollutant accumulation, followed by the rapid mobilisation of the accumulated material [*Mansel*, 2001].

Wash-Off

The wash-off process begins when erosive agents such as wind and water detach particles from surfaces. Rain splash can be the main agent in detachment as individual grains are splashed as high as 60cm into the air and 150cm laterally [*Schwab*, 1981]. The particles are then free to be transported by runoff. It is thought that the greatest transport of sediment and sediment-associated pollutants occurs within the first few moments of runoff production. Maximum pollutant concentrations are reported to occur in the first 12-15mm of runoff, with much lower concentrations occurring afterwards [*Barbosa and Hvitved-Jacobsen*, 1999]. This phenomenon is known as the first flush and is discussed in more detail below.

The 'first flush' is a phenomenon associated with sediment wash-off during rainfall events. It is defined by [*Gupta*, 1996] as "the initial period of stormwater runoff during which the concentration of pollutants is substantially higher than those observed during the later stages of the event". It has been suggested that a first flush has occurred when the first 50% of the runoff volume for each event transports over 50% of the total suspended solids [*Delleur*, 2001]. The occurrence of a first flush is of critical importance to retention pond water quality since a substantial concentration peak at the beginning of storm events results in much of the available sediments being washed off in the first 1-2mm of runoff. This may produce 'shock loadings' that are detrimental to aquatic life. It is for this reason that stormwater management basins in the U.S. and Canada are designed to capture the first 0.5-1 inch (12.7- 25.4mm) of runoff [*Pitt*, 2005].

The occurrence of a first flush is highly controversial, and inconsistent results are reported in numerous studies investigating this phenomenon [*Deletic et al.*, 1997; *Delleur*, 2001; *Ellis and Hvitved-Jacobsen*, 1996] although perhaps the inconsistency in results can be attributed to the numerous and varied definitions used to describe the first flush, as well as the variation in field techniques and sites used by different researchers.

2.1.4 Sediment Particle Size, Distribution and Water Quality

Particle size is a key determinant of water quality [*Schroeter and Watt*, 1989]. It is widely accepted that smaller particles such as clay have a greater charge and a relatively larger surface area for adsorption. This encourages metals, bacteria, oils and other pollutants to bind to them [*Ellis and Hvitved-Jacobsen*, 1996]. The literature suggests that a significant proportion of the total and toxic polluting load arising from urban surfaces is associated with the fine sediment fraction (<63μm diameter) [*Charlesworth and Lees*, 1999; *Ellis*, 2000], with some authors stating more specifically that a high proportion of the contaminants are associated with the very fine particles of less than 2μm [*Greb and Bannerman*, 1997]. Other studies undertaken to determine the metal concentration associated with different particle sizes have shown a consistent trend. For example, a metal analysis of sediment particles from snow and rainfall events by [*Sansalone and Buchberger*, 1997b] indicated that adsorbed Zn, Cu and Pb mass increased with decreasing particle size. Furthermore, findings from an analysis of heavy metal loadings in stormwater samples from Toulouse, France, showed that the largest proportions of metals were associated with particles less than 10μm in size (Vignoles and Herremans, 1995).

Contrary to the bulk of research concerning particle size and contaminants, a study by [*Stone*, 1996] showed that a high proportion of heavy metal mass was associated with the coarser particle fractions. This relationship has also been found in a few other studies, for example [*Hepburn*, 2004], which found a high proportion of heavy metal mass to be associated with the fine fraction as well as with the coarse fraction. The implication of such findings should be considered in terms of stormwater management since most of the research to date (and thus many of the stormwater management techniques) are concerned with reducing influent particles of less than 63μm, since it is traditionally the fine fraction which is thought to be associated with a high proportion of the contaminant load. The consequences of high metal loads being associated with larger particles could, however, be beneficial for pond water quality performance, since coarser material is easier to remove from a pond than the fine grained material.

Summary

In summary, sediment is the major vector in most urban contaminant transport because it provides the binding site to which pollutants, heavy metals and hydrocarbons adhere and once in the water environment, they can persist for long periods of time. This is particularly true of the finer fraction which strongly adsorbs pollutant particles. There are a number of important processes involved in sediment-pollutant transport which result in the movement of contaminant particulates from source sites such as highways, residential, commercial and industrial areas into the drainage systems of urban developments. Such particulate matter can often be highly toxic and ultimately finds its way into the aquatic environment.

2.2 Sustainable Urban Drainage Systems (SUDS)

SUDS are drainage systems designed to reduce flood risk in the catchment and protect downstream watercourses from water quality deterioration. Although there are several types of SUDS, nearly all function using the principle of source control, a pre-emptive strategy that aims to intercept stormwater (and its associated pollutants) at source and dispose of it close to the point of rainfall, thus returning catchment flows to their pre-development rates. This is achieved using structures which attenuate flows, improve water quality and dispose of runoff close to source [*Ellis*, 2000].

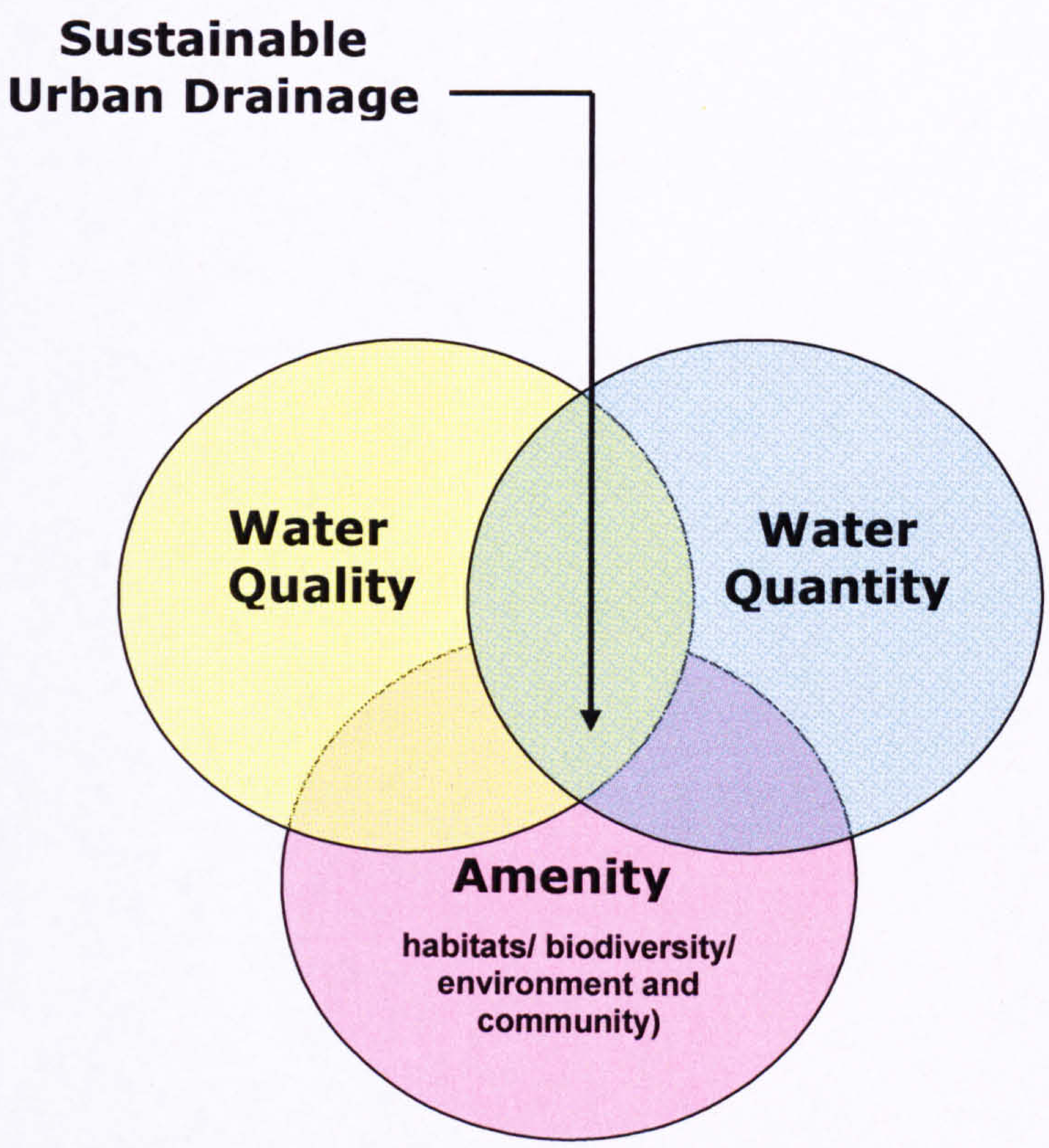


Figure 2.3: SUDS sustainability triangle illustrating the holistic approach to drainage (adapted from SUDS Working Party, 2001).

Drainage systems can be designed to meet the ideals of sustainable development by providing a holistic approach which integrates the three domains of the SUDS sustainability triangle: water quantity, water quality and amenity issues in the local community (see Figure 2.3). To ensure these ideals are met, SUDS should be used alongside a number of 'good housekeeping' strategies, such as street sweeping to remove sediments and litter from urban surfaces prior to storm events. Good housekeeping measures should, like all SUDS approaches, aim to satisfy the "principle of subsidiarity", which is to manage any issue as close to source as possible [*CIRIA*, 2000]. In satisfying this single principle, runoff will be returned to the natural drainage system as close to its source as possible, mirroring the natural or pre-developed behaviour of the catchment. A number of different SUDS can be designed to function singly or together as part of a sustainable urban drainage treatment train (Figure 2.4). This provides treatment at different scales including source control, site control and on a larger scale, regional control, to ameliorate, to varying degrees the effects of urbanisation on watercourses [*CIRIA*, 2000].

A number of political drivers are encouraging the use of SUDS systems on new urban developments. In particular the Water Environment and Water Services (Scotland) Act (2003), referred to hereafter as WEWS (2003), defines SUDS and sets out guidelines for their maintenance. WEWS (2003) sets out a number of Controlled Activities Regulations (CARegs), in direct relation to surface water. Schedule 3 and the General Binding Rules 10 and 11 contained within it set out specific rules in relation to (i) discharge of runoff from a surface water drainage system to the water environment from construction sites, buildings, roads, yards or any other built developments and (ii) discharge into a surface water drainage system (Neil McLean, Personal Communication). The rules set out for (i) and (ii) aim to protect both SUDS devices and the watercourses they discharge into (Appendix A).

There are four main types of SUDS: swales, permeable paving, infiltration devices and basins. These are each described in more detail in the following sub-sections.

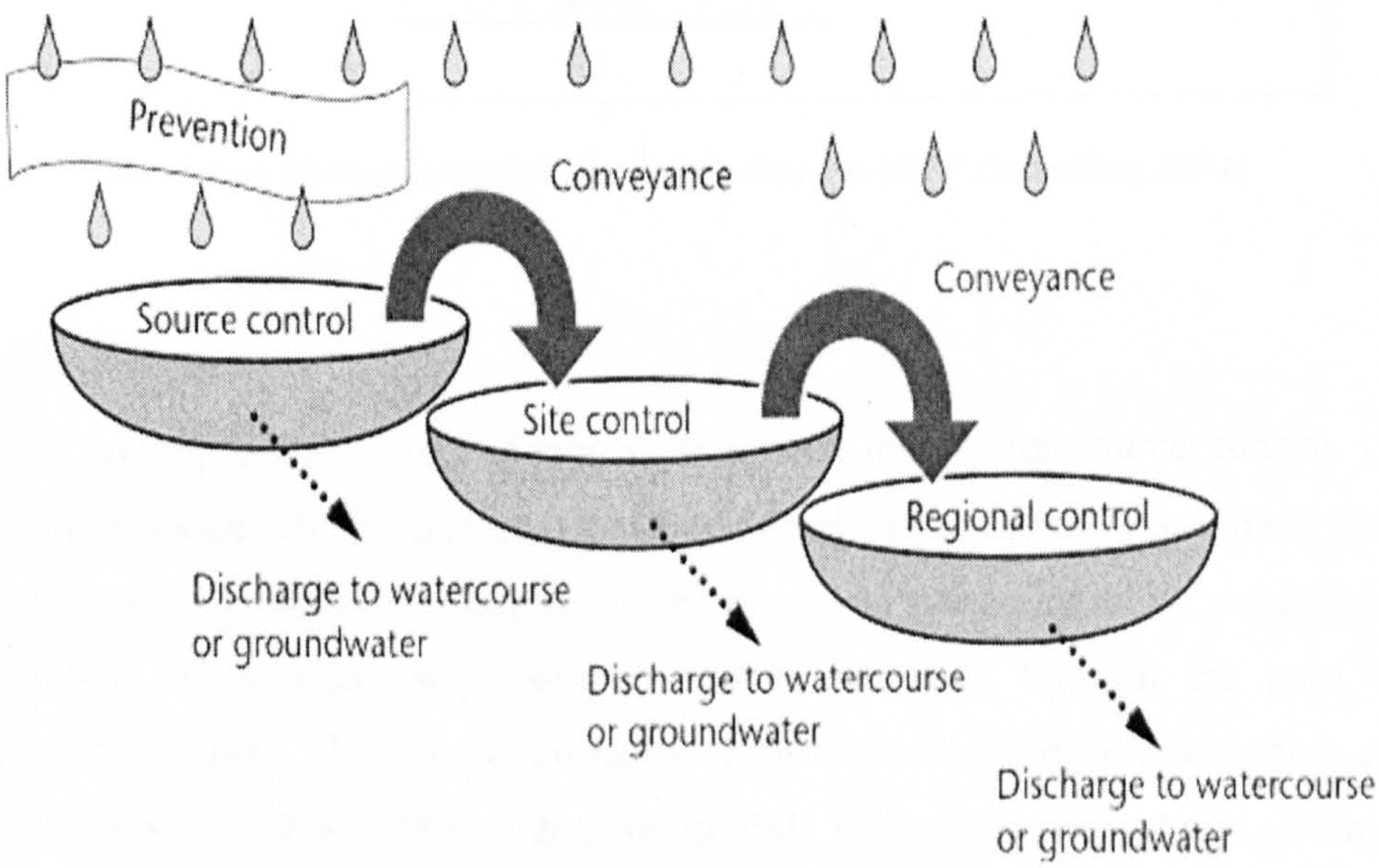


Figure 2.4: Diagram of a treatment train incorporating source, site and regional control (Source: Environment Agency for England and Wales, 2004).

Swales

Swales are devices that take the form of a shallow, vegetated channel or trough, often used to convey water to a pond or wetland prior to discharge to a watercourse. In the process of transferring water, their wide-open nature allows water to infiltrate to the ground, slowing runoff velocities, while pollutants are removed by filtration and microbial decomposition in the humic root zone of the soil [*CIRIA*, 2000; *Mikkelsen et al.*, 1996; *SEPA*, 2003]. Figure 2.5 shows a cross section through a typical roadside grass swale.

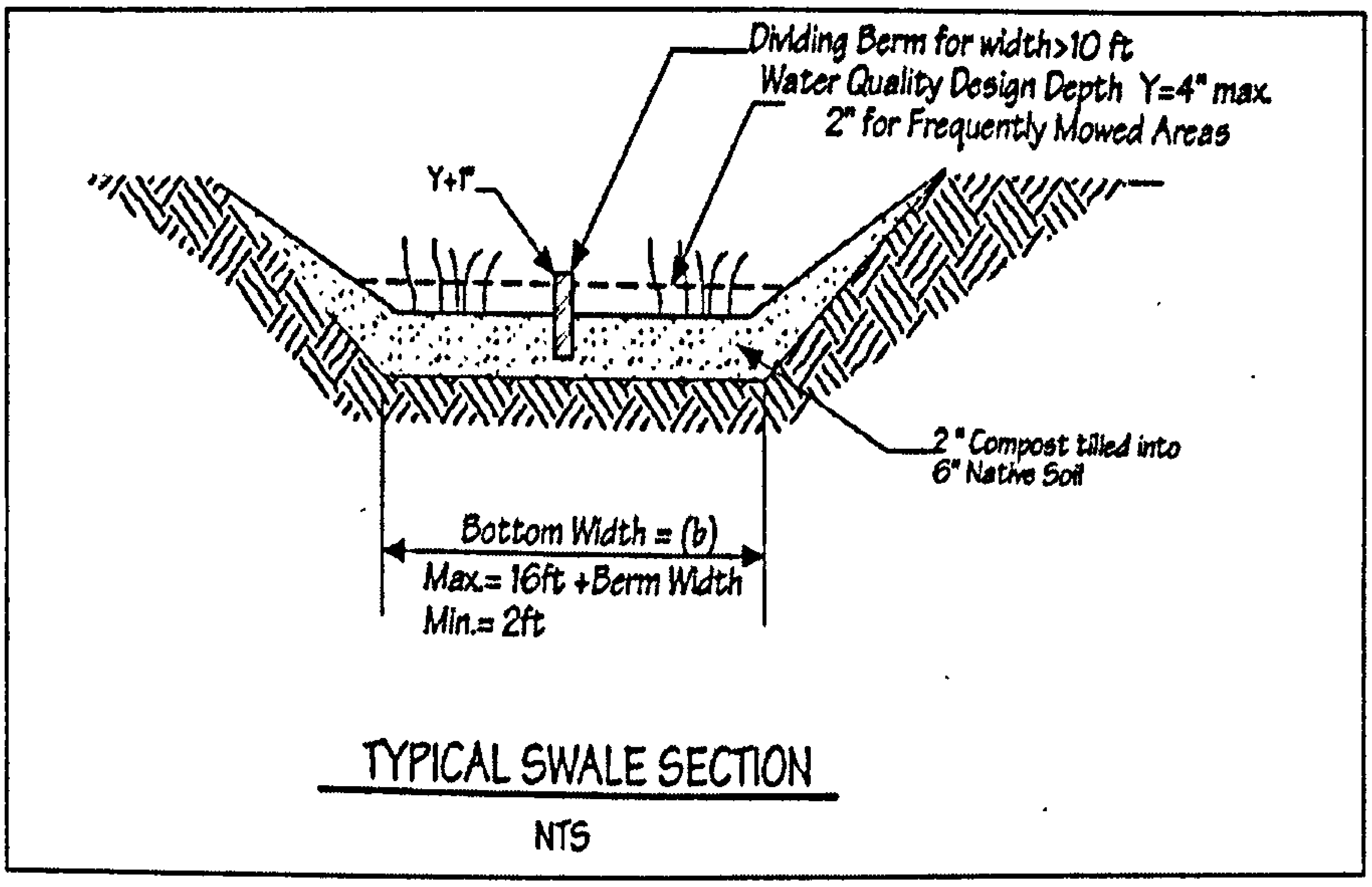


Figure 2.5: Cross section through a typical grass swale Source: [*RBF Consulting*, 2001]

Permeable Paving

Permeable paving is a surface drainage system used mainly for source control, typically made from pavement blocks, porous asphalt or gravel. Although there are many variations, typically the paving blocks are designed to be porous in nature, to allow water to infiltrate through them, while also encouraging runoff to permeate through the gaps between individual block units. This is in contrast to conventional paving blocks which are often completely impermeable. They intercept rainfall at source, and where conditions are appropriate, water may infiltrate directly into subsoil, or alternatively, be held in a sub-paving reservoir for delayed discharge into another structure (Figure 2.6). Recent research has shown that permeable paving can reduce stormwater peak discharges substantially, in many cases to zero [*Stormwater Industry Association*, 2003]. Pollutants are held in the subsurface and are filtered down through the subsoil [*SEPA*, 2003]. These SUDS are most widely used in connection with car parks, patios and driveways in residential areas [*CIRIA*, 2000].

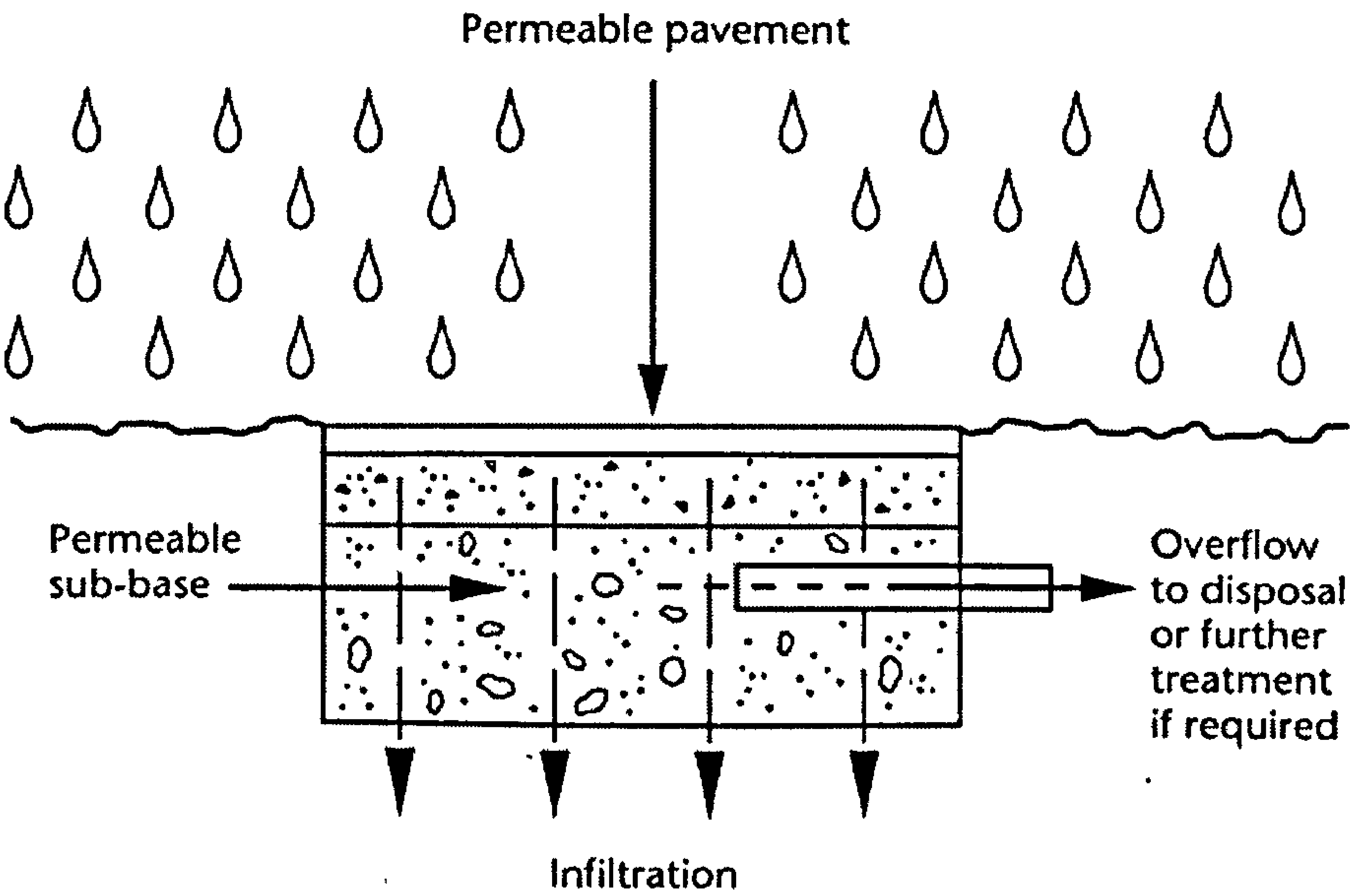


Figure 2.6: Cross section through permeable paving layers and sublayers [*EA*, 2004].

Infiltration Devices

Infiltration devices, such as filter drains and soakaways are much more common in Scotland than surface systems, although they are less prominent than ponds and wetlands, in terms of the catchment area served [*Sniffer*, 2002]. They are generally used for site control and, although there are many different infiltration techniques, all are based on the principle of filling a subsurface volume (such as a trench) with gravel or crushed stone to create an underground reservoir to temporarily store stormwater. The runoff percolates into the subsoil gradually or is discharged to another structure at a controlled rate. As well as allowing water to dissipate gradually, pollutants are held close to, or in the trench where filtering and decomposition occur [*CIRIA*, 2000; *Mikkelsen et al.*, 1996]. Figure 2.7 shows a cross section through an infiltration trench, one such type of infiltration system.

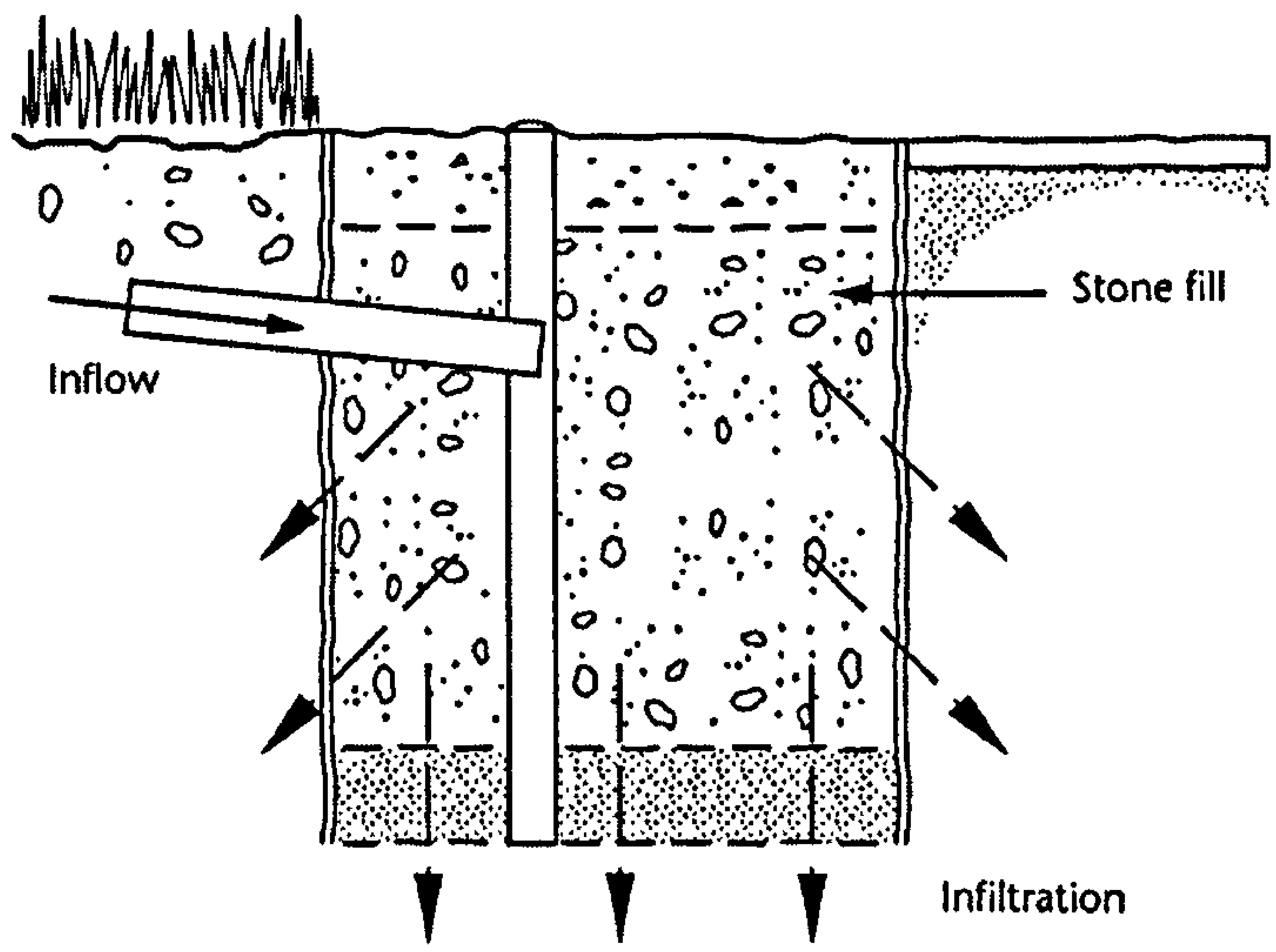


Figure 2.7: Cross section through an infiltration trench [*EA*, 2004].

Infiltration systems have an extremely high rate of failure, often due to clogging by fines. For example, infiltration trenches have a 1 in 2 risk of failure over the first five years. In trench systems in the US mid-Atlantic region failure was as high as 50% and there have been similar findings with European infiltration devices. In Copenhagen, clogging due to fine particles reduced the hydraulic conductivity by 30-70% and accounted for failure rates of 2.6 times per year over a five year period [*Warnaars et al.*, 2000].

A further concern regarding infiltration systems is the potential threat to groundwater quality [*Barbosa and Hvitved-Jacobsen,* 1999]. There is a high risk of leaching, and thus a release of mobile species into the unsaturated zone beneath infiltration systems [*Ellis*, 2000; *Mikkelsen et al.*, 1996]. Research from a study of a soakaway in a 26 ha residential site in Luton by [*Ellis*, 2000] highlighted the potential for metal leachates to be mobilised towards the unsaturated zone. The results showed that heavy metal concentrations of Zn, Cu, and Cd increased annually with increasing depth in the trench and highlighted the potential for soluble species to be mobilised into the underlying unsaturated zone.

Stormwater Management Basins

Stormwater management basins are regional stormwater management systems that can be used for the dual purposes of reducing flood risk and improving water quality. However, different types of basin are designed to provide different benefits. Basins designed to decrease post-development peak discharges to less than or equal to those which occurred before urbanisation, attenuate flows by providing storage capacity during storm events [*Butler and Davies*, 2000]. Basins designed to improve water quality do so by the provision of long residence times and quiescent hydraulic conditions that promote the settling of contaminated sediment. Further water quality enhancement can be gained by the incorporation of bank-side and aquatic vegetation that aids the removal of dissolved nutrients and contaminants by biological uptake and microbiological decomposition [*Campbell*, 2004; *Marsalek and Chocat*, 2002]. The research in this thesis is mainly concerned with the performance of a specific type of basin, namely retention ponds. Since the sizing and design of retention ponds is generally based on the same principles that are used for detention basins, both basin types are described in full below.

2.3 Design of Detention and Retention Ponds

Detention basins and retention ponds are designed for different purposes. Detention basins are designed primarily for flow attenuation, although there is some evidence that water quality may be enhanced, whereas retention ponds are designed to provide both water quality enhancement and flow attenuation.

In the past, the design of stormwater management basins has occurred with a strong emphasis on flood detention, and thus stormwater detention for flood protection is relatively well understood [*CIRIA*, 1993; *Rowney*, 1986]. However in recent years, there has been a shift in basin design, reflecting concerns regarding the protection and improvement of water quality. According to the principles of sustainability, SUDS should not only be sustainable in terms of the way they provide water quality and quantity control, but they should provide amenity for the local community and habitat for wildlife.

Detention Basins

Detention basins (Figure 2.8) are predominantly dry, except during storm conditions when they collect and store runoff temporarily. They contribute significantly to reducing peak flows by providing an area of temporary storage during storm events, however they have lower pollution removal capabilities than retention ponds because particle settling can only occur when there is a temporary pool of water. Since this only occurs temporarily during rainfall events they have relatively short hydraulic residence times. They may also be susceptible to the resuspension of previously settled particles [*Butler and Davies*, 2000; *CIRIA*, 2000].

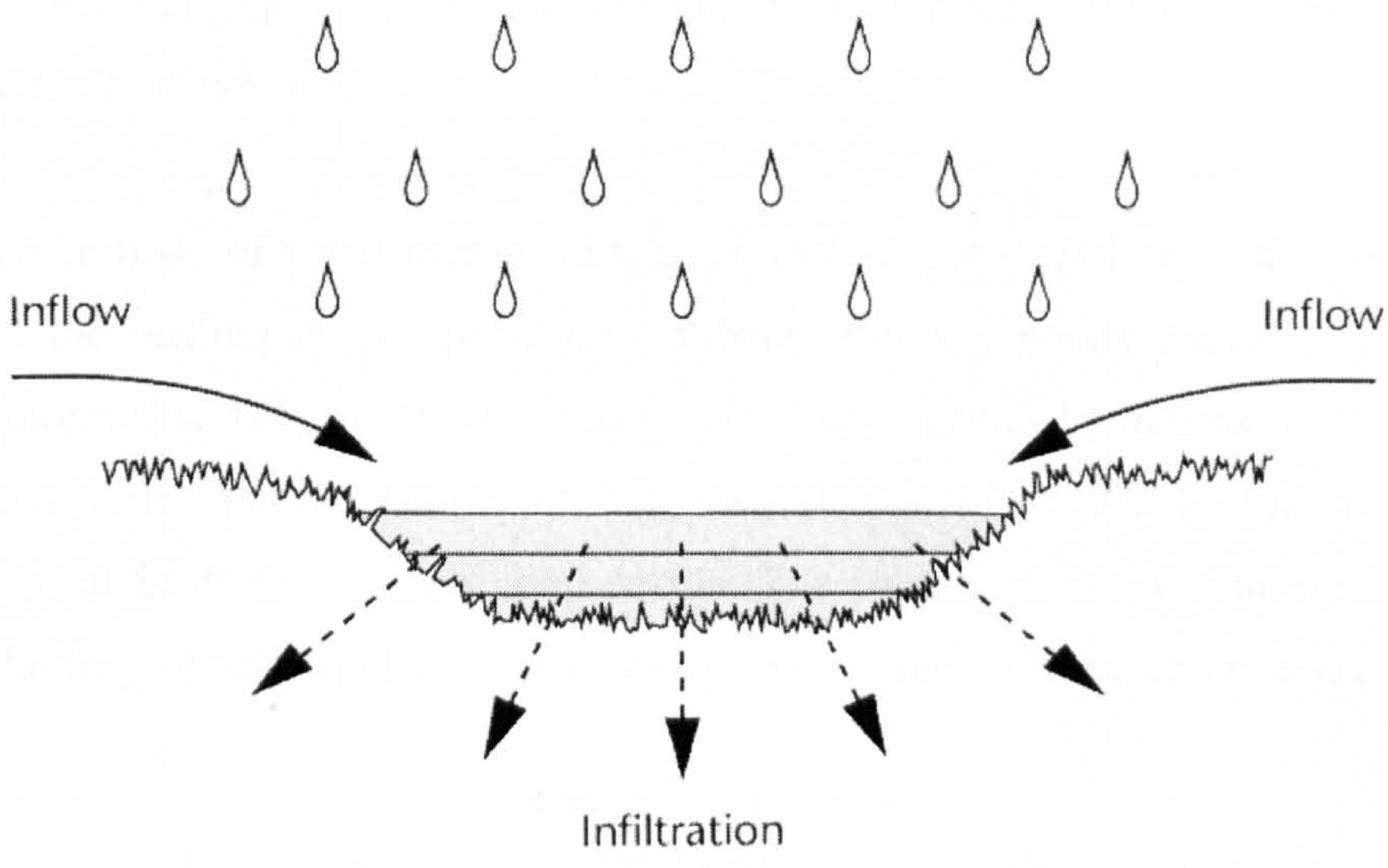


Figure 2.8: Cross section through a basin during storm conditions (Source: EA, 2004).

Despite much of the early literature describing a negligible role for detention basins in sediment removal, studies from various countries report deposition of sediments in basins [*Lovern*, 2000; *MUDWADE*, 2000; *Pettersson*, 1999a; *Sniffer*, 2001; *WRRI*, 1989]. For example, in the UK, visual inspection at the Dunfermline Eastern Expansion Project (DEX) site, Scotland, has identified the deposition of sediments ranging in size from coarse material to fine silts in dry detention basins [*Sniffer*, 2001].

Compared to retention ponds, detention basins do not make a contribution to community amenity. However, since they are predominantly dry, there are few health and safety risks associated with them.

Retention Ponds

Retention ponds are stormwater management basins, which have a permanent pool of water. They attenuate flows by providing an area of temporary storage during storm events, which helps reduce storm peaks and delay outflow. Their ability to attenuate flows is dependant upon the storage volume available in the pond and the magnitude of the storm, as well as the varying characteristics of the sequence of storms they are subject to throughout their design life.

The permanent body of water present in retention ponds provides long residence times and encourages the settling out of pollutants. Where retention ponds provide an ecological habitat, biological activity (such as the uptake of metal species by aquatic vegetation) may also improve water quality [*Butler and Davies*, 2000; *CIRIA*, 2000]. Figure 2.9, below shows a typical Scottish retention pond design. The aquatic plants, visible in the diagram, illustrate the ways in which SUDS basins can provide habitat enhancement and amenity.

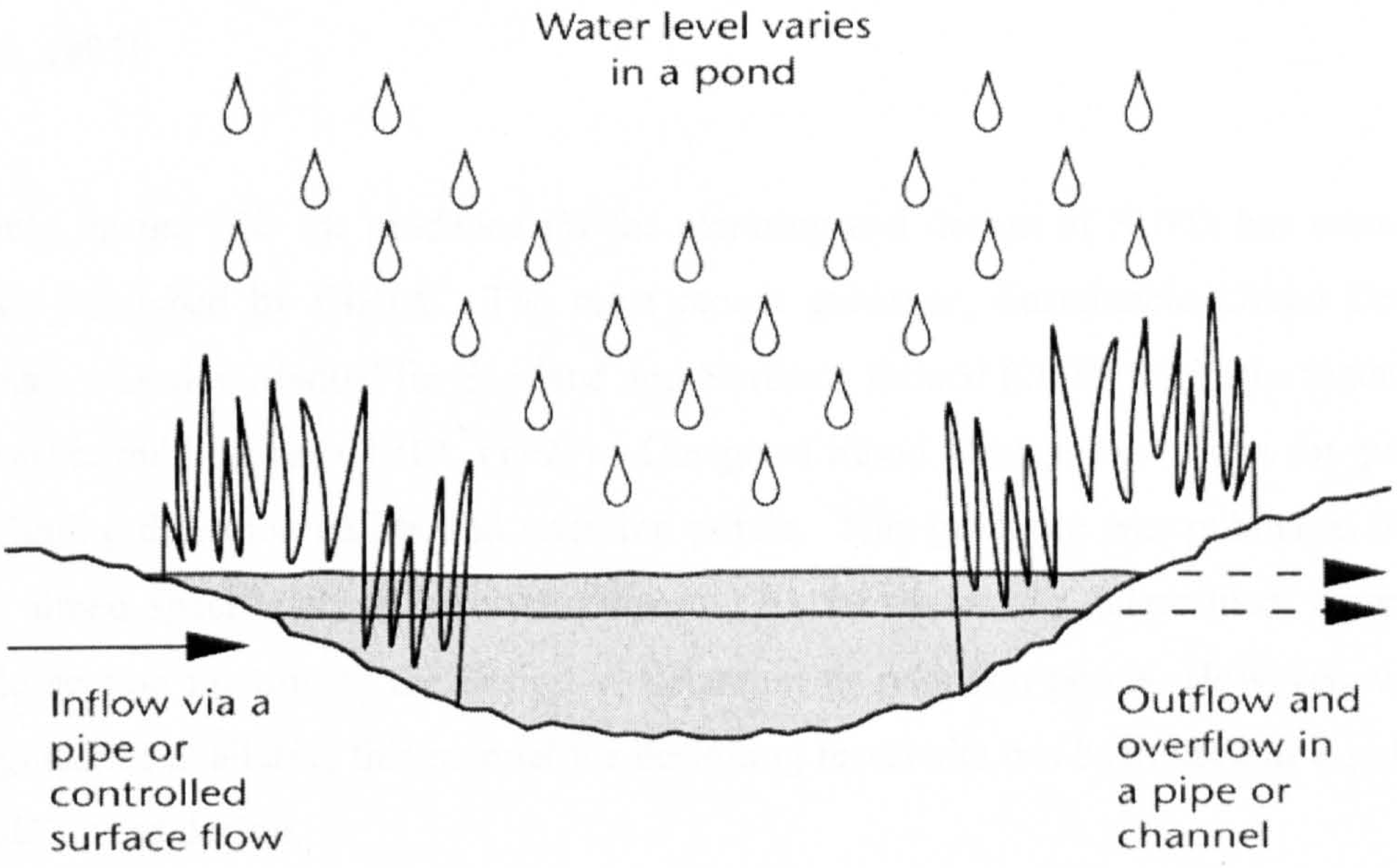


Figure 2.9: Cross section through a retention pond (Source: EA, 2004).

Retention ponds embrace all of the principles of the SUDS sustainability triangle by improving water quality, reducing flood risk and by making a positive contribution to community amenity, and in many cases, providing ecological habitat. This often makes retention ponds the preferred sustainable drainage solution and is perhaps why they have been applied successfully worldwide, in countries such as Australia, Canada, USA, Sweden and Denmark [*Marsalek and Chocat*, 2002]. However retention ponds, have met with some criticism particularly where they have been installed into existing residential areas. This criticism has mainly been due to residents' fears over safety.

2.3.1 Stormwater Basin Design for Flow Attenuation

Detention basins and retention ponds must be sized to provide adequate flood control. Furthermore, they must be effective at detaining and retaining a storm event of a very rare frequency (e.g. 100 year storm) to protect against flooding, whilst also being able to manage the smaller events - which are much more important in terms of water quality improvement [*Hingray*, 2002; *Pitt*, 2005]. Flood protection requirements vary depending on the catchment size, extent of urbanisation, anticipated future urbanisation and the existing hydrological regime in the catchment, as well as a number of engineering constraints such as the size of the area available for the basin, and the available budget [*CIRIA*, 1993].

Currently, in the UK, the guidance for the planning and design of SUDS has come from manuals published by CIRIA. The most recent guidance, Sustainable Urban Drainage Systems – a Design Manual for Scotland and Northern Ireland (2000) refers the reader back to an earlier publication, CIRIA, (1993) – Design of Flood Storage Reservoirs for guidance on designing detention basins and retention ponds. This guidance was published in 1993 and is aimed specifically at reservoir design. As far as can be determined, there is no specific section relating to the design of detention or retention ponds. However, with no other guidance available, this manual for designing reservoirs has been used as a guideline for SUDS pond design.

Before detailed design can begin, a suitable design standard must be selected. According to [*CIRIA*, 1993], the standard to which the system is designed will be entirely dependant upon the policies of the particular local water management authority. However, as a general rule the design standard may be:

1. The average interval between exceedences of a given flow released from the system, or
2. A return period based on the channel capacity and flood levels downstream.

The principal design criteria are the critical storm and the storage volume required to provide flood attenuation, or the amount of attenuation associated with a chosen storage volume. Inflow hydrographs, specific to the planned location of the basin, are required to enable the critical storm to be determined. The critical storm is calculated from a range of storm durations that have the same return period as the chosen design storm. The hydrographs are routed through the reservoir to provide an indication of maximum storage volume required and probable maximum and minimum water levels. Finally, confirmation (from ground levels and contours) that the maximum storage is available at the chosen site is required. When this data has been gathered, environmental consultations with appropriate stakeholder organisations are required to ensure there are no objections to the proposal and to enable important environmental issues to be considered.

Figure 2.10 illustrates the recommended stages involved in conducting flood estimation for a catchment in the planning of reservoir (and detention basin) construction.

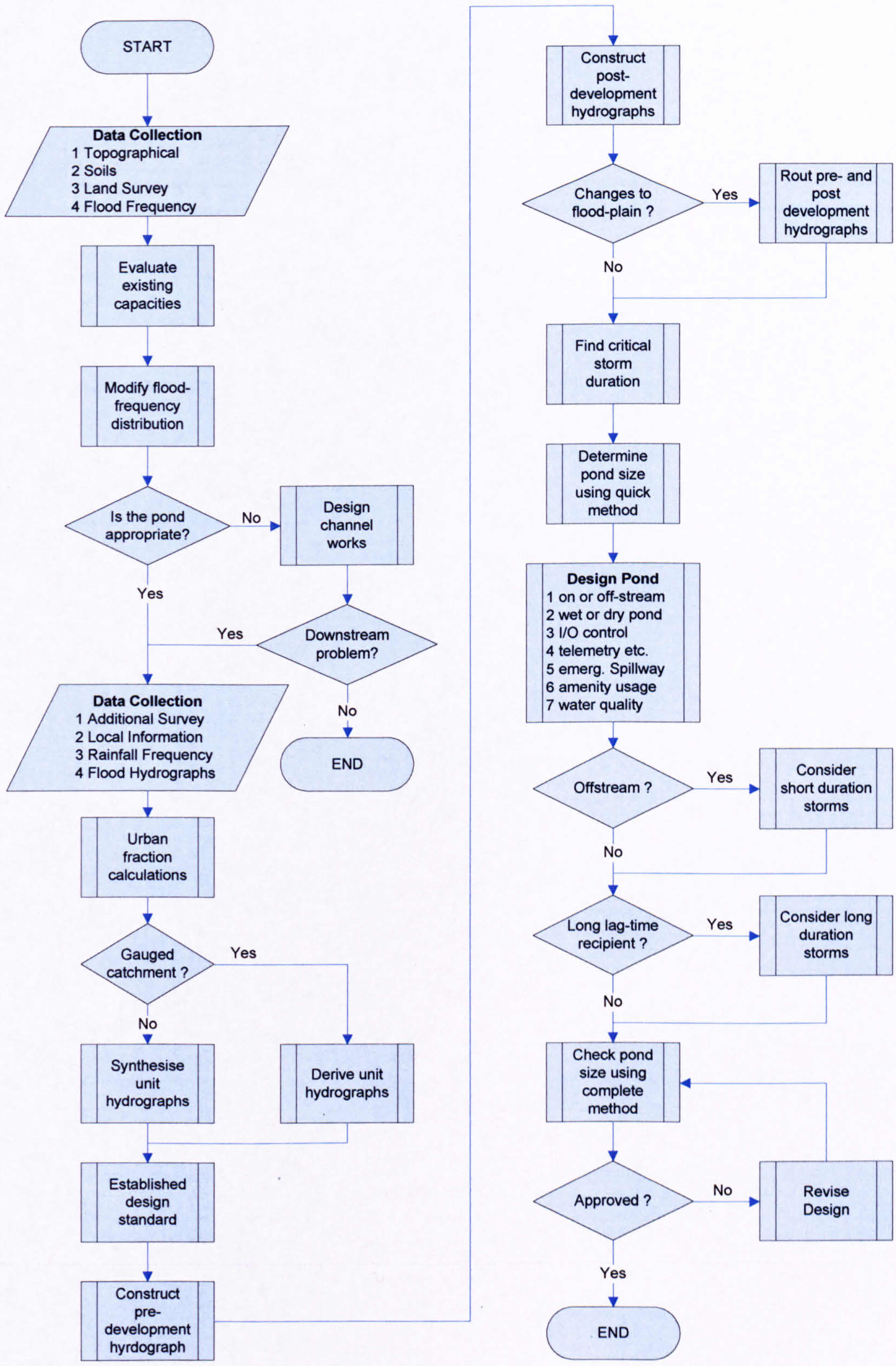


Figure 2.10: Flow Chart illustrating the steps involved in designing a detention basin.

Detention basins in the UK are designed using the approach in Figure 2.10 to provide a specific level of flood storage. Often basins are sized in terms of their plan area in relation to the area of the contributing catchment. In other countries, the design storms chosen for flood control are usually specified by regulation guidelines or are based on critical catchment characteristics [*DCR*, 2002]. An assessment of groundwater conditions and the infiltration rate through the pond sides and bottom are also important in the determination of flood control volume. Fluctuations in groundwater levels can temporarily reduce storage volume, cause slope failure and affect a pond's ecological functions [*France*, 2002].

In terms of designing for flow attenuation, retention ponds are based on the same criteria as detention basins, however, they are designed to have a dry storage volume above a permanent pool area so that they can reduce peak flow rates. In urban areas in the USA, this is usually sized for 2-year, 10-year and 25-year return period storm events [*DCR*, 2002], while guidelines in the UK design retention ponds to reduce peak flows for the 10, 25 and the 100 year flood events [*CIRIA*, 1993]. A review of stormwater basin design guidance available in the UK and the US conducted for this chapter has highlighted the common technique of designing stormwater management basins to attenuate a one-off storm event of a specified magnitude and taking no account of prior storm events (the critical storm). The drawbacks of this approach are explored in Chapter 4 of this thesis; in practice the flow attenuation performance of a retention pond is determined by the pond's ability to attenuate a sequence of storm events of varying magnitude.

In general, basins of both types are designed to reduce *peak flow rates* and return them to pre-development rates, thus helping to reduce flood risk in the immediate vicinity of the basin and in the areas directly downstream. However, it must be born in mind that SUDS basins do not mitigate the increases in *runoff volume* experienced in the urban catchment to the pre-development situation (unless infiltration occurs). This means that in many cases, all of the pond inflow volume is eventually passed downstream, where channels will inevitably experience higher depths of flow for longer periods, and may increase coincidence with other flow peaks in the catchment, thus increasing flood risk. The new flow regime in downstream watercourses may affect slope stability in channels and change downstream flow characteristics in comparison to pre-development flows [*Glazner*, 2001]. These downstream effects are not considered within this thesis.

Evidence of Flow Attenuation in Stormwater Management Basins

Despite many studies confirming that SUDS basins perform well in reducing peak runoff rates [*Butler and Davies*, 2000; *Glazner*, 2001; *Yeh and Labadie*, 1997] thus decreasing flood risk in the catchment, there is a lack of studies that quantify the level of flood risk reduction provided by SUDS basins, or which confirm that basins return flows in the catchment to those experienced before development took place. A large number of studies of stormwater management basins have been conducted in the USA, however, most focus on water quality issues, i.e. sediment and pollutant removal efficiency, as opposed to flow attenuation performance [*Pratt*, 2001].

[*Johnston*, 2000] investigated the efficiency of four retention ponds in reducing flood risk on the Dog River Catchment, Alabama, USA. These ponds were assessed for their ability to attenuate storm events at the 5, 10, 50, and 100-year storm return periods. The results showed that although the ponds were able to perform adequately for the 1 and 2-year rainfall event, they failed to retain flows sufficiently for events greater than a 5-year event, resulting in overflow. It was concluded that increasing the surface area or the depth of the ponds would increase the flow attenuation. Unfortunately, it is unclear from this study what return period the retention ponds had been designed to attenuate, however BMP guidance manuals suggest that retention ponds in densely urbanised areas of the USA (such as the Dog River Watershed) are designed to attenuate flows to at least the 25-yr storm event.

Likewise, modelling the ability of four retention basins to reduce peak flows by [*Hingray*, 2002] in a Swiss urban catchment showed that the basins, although able to retain flows for storm events with return periods between 10–50 years, were unable to reduce peak flows for events approaching the 100-year return period. According to [*Johnston*, 2000], it is a common occurrence for retention ponds to be unable to attenuate large events.

In the UK there are no long-term studies that asses the performance of stormwater management basins in critical flow events. Two retention ponds at the DEX site, in Dunfermline, Scotland, have been monitored over a 2-year period. Both Halbeath and Linburn ponds achieved a reduction in peak flow, however, Halbeath pond appears to perform considerably better in terms of peak reduction and flow attenuation with its outflow hydrograph showing consistently lower peak flows per hectare catchment area [*Sniffer*, 2001]. Again, it is unclear from the study what return period storm the ponds have been designed to attenuate, however, since the study is conducted over a relatively short time, it is difficult to make any inference about the long-term performance of the ponds.

Summary

Stormwater management basins can be effective in reducing flood peaks, particularly for storms with small return periods. However evidence suggests that retention ponds do not consistently provide flood peak reduction for storms of a large recurrence interval (5-100 years). This may be due to the nature of pond functionality which aims to provide both the competing functions of flood protection and water quality, or as discussed later in Chapter 4, it may be due to poor pond planning and design. This thesis considers the factors that influence good flow attenuation performance in stormwater retention ponds with the aim of determining the optimum pond design for effective flow attenuation. Note that the effects of storm magnitude, frequency and storm sequence are of particular significance in this regard.

2.3.2 Stormwater Basin Design for Water Quality

Treatment is primarily achieved in stormwater management basins by encouraging the settling out of sediments. Long periods of quiescent hydraulic conditions are therefore required to promote the settlement of the fine grained particulates, which are usually associated with the highest pollutant loads. Such conditions can be achieved in a number of ways as discussed in section 2.4.

There have been concerns over the ability of SUDS basins to remove the most polluted component of the runoff. Research has shown that while systems are able to trap the coarsest fractions, they may not be removing enough of the fine fraction to prevent contamination of receiving waters. Both [*Farm*, 2001; *Greb and Bannerman*, 1997] illustrate the strong relationship between sediment (and associated pollutant) removal efficiencies and influent particle size distribution in stormwater management ponds. The ponds studied were highly efficient in removing sand and silt particles but removed only a limited proportion of clay-sized particles. It is suggested that higher success rates in controlling the fate of particles smaller than 2μm would greatly improve the effectiveness of urban ponds, since it is these smaller particles that are normally associated with the high concentration of polluting material [*Farm*, 2003; *Greb and Bannerman*, 1997; *Vignoles*, 1995].

Detention basins are not specifically designed to provide water quality treatment but, as previously discussed in section 2.3.1, there is evidence that sediments are removed from stormwater in a number of these systems. Retention ponds, on the other hand, are designed *primarily* to improve water quality and this is achieved by promoting quiescent conditions that encourage contaminated sediment particles to settle out of suspension. Although water quality can also be improved by bio-chemical processes only water quality enhancement by the mechanisms of sedimentation and dilution is considered in this thesis.

Assessment of Water Quality Performance

The water quality performance of a pond is often a measured by its removal efficiency for a range of pollutants commonly present in stormwater runoff (e.g. suspended solids, heavy metals, nutrients, dissolved material). In much of the literature the concentration of pollutants in stormwater management basins is described by the Event Mean Concentration (EMC), which can be determined using equation 2.1. This is defined as the mass of pollutant transported in a runoff event divided by the volume of water in the event. EMCs can either be collected for an individual event or for a set of field measurements which can then be used to calculate annual and extreme values [*German*, 2003].

$$\mathrm{EMC} = \frac{\sum_{i=1}^{n} ViCi}{\sum_{i=1}^{n} Vi} \qquad \text{Equation 2.1}$$

Equation 2.1: Where *Vi* is the volume of flow during the inflow period *i*. *Ci* is the average concentration associated with period *i*, and n is the total number of measurements taken during the event.

In much of the published research, pollutant removal efficiency is evaluated by placing a water sampler at the inlet and one at the outlet. The removal efficiency is determined by assessing the difference between the pollutant concentrations at the inlet and the outlet for a storm event. However, [*Pettersson*, 1999a] caution against the use of single event observations to make inferences about long-term removal efficiencies since this can lead to very misleading results for some contaminants.

An alternative method used by [*Pettersson*, 1999a] to calculate EMCs monitors pollutants in several successive storm events using flow-weighted samples. The total amount of pollutants yielded at a monitoring location is calculated by multiplying the concentration of each sample by the associated stormwater volume from the storm hydrograph. The mass of each pollutant is then summed to give a total polluting mass for all pollutants during the storm. EMCs can also be calculated as the ratio of total pollutant load to stormwater volume.

Table 2.2 has been compiled for this chapter from all the literature available that reports the efficiency of retention and detention basins for removing typical urban stormwater contaminants. The efficiency for both basin types in removing contaminants from stormwater is highly variable, ranging from 30%-70% removal efficiency for some contaminants such as Nitrogen in retention ponds, and 42-92% removal efficiency of Total Suspended Solids in detention basins. Removal efficiencies shown in Table 2.2 suggest that in many cases (e.g.Total Suspended Solids and Suspended Solids) detention basins are potentially as effective at pollutant removal as retention ponds. However, retention ponds clearly perform better in removing dissolved nutrients (e.g. Nitrogen and Phosphorus) as well as heavy metals (e.g. Zinc). While much of the design guidance available suggests that retention ponds should remove a significant percentage of the polluting load, it has been, to date, widely accepted that detention basins offer little or no water quality improvement. Table 2.2 demonstrates a growing body of literature that indicates that detention basins may offer a reasonable contribution to water quality improvement. Furthermore Table 2.2 shows that a well-designed retention pond can achieve removal efficiencies of over 90% for solids, however, they do not always meet their design specification, and there are many instances where pollutant removal is very low (e.g. Total Suspended Solids removal of 26%). It should also be noted that the material washed into SUD systems are not wholly inorganic particles and may be composed of a number of substances of varying density.

It has been suggested that there may be a limit to how much water quality can be improved before discharge to a watercourse. A number of studies propose that pollutant concentrations within SUDS cannot be reduced below a baseline value, referred to as the irreducible pollutant concentration, because of limitations in removal pathways and internal production by plants and microbes [*Kadlec and Knight*, 1996]. However, as can be clearly seen from Table 2.2, there is enormous room for improvement.

Table 2.2: Minimum and maximum removal efficiency (as a % of mass removed) by stormwater basin for a number of typical stormwater contaminants

Constituent	System Type	Removal Efficiency (%)	Reference
Total Suspended Solids	Retention Pond	26-92	[*Farm*, 2003; *Luzkow*, 1981; *Mead*, 2003; *Pettersson*, 1996; *Pettersson*, 1999a; *SWAMP*, 1998; *USEPA*, 1987]
	Detention Basin	42-92	[*Mead*, 2003; *USEPA*, 1987]
Suspended Solids	Retention Pond	91-95	[*Striegle.*, 1987]
	Detention Basin	50-90	[*DCR*, 2002] MWCOG(1992 , in DCR)
Dissolved Solids	Retention Pond	80-90	[*Hartigan*, 1989]
	Detention Basin	40-80	MWCOG (1992 , in DCR)
Total Phosphorus	Retention Pond	39-79	Luzcow et al., (1981); Gietz (1983); USEPA (1987); Hartigan, (1989); SWAMP, (1998); Kadlec and Knight, (1996)
	Detention Basin	20-30	Hartigan, (1989)
Total Nitrogen	Retention Pond	30-70	USEPA (1987); Hartigan, (1989)
	Detention Basin	20-30	Hartigan, (1989)
Lead	Retention Pond	50-94	Striegl, (1987); Pettersson, (1996); Pettersson, (1999)
	Detention Basin	70-80	Hartigan, (1989)
Zinc	Retention Pond	42-94	Striegl, (1987); Stanley (1996); Kadlec and Knight 1996
	Detention Basin	40-50	Hartigan (1989)
Biological Oxygen Demand	Retention Pond and Detention basin	20-70	USEPA (1987); Hartigan (1989)
Chemical Oxygen Demand	Retention Pond and Detention basin	20-70	USEPA (1987); Hartigan (1989)

2.4 Factors Affecting Pond Water Quality Performance

Once in a pond, sediments are subject to a number of physical processes which govern their behaviour. According to France (1999) the three most important factors affecting particle settlement for water quality enhancement are a) particle size and distribution (and thus the settling velocity), b) residence time and c) particle resuspension. The following sections describe each of these mechanisms in turn.

According to [*Pettersson*, 1999a] the pollutant removal efficiency varies for different ponds due to differences in the ratio of pond surface area to impervious catchment area (SAR ratio). Furthermore, retention ponds can require around three to six years to establish an ecological balance. During this period of establishment ponds may experience nutrient imbalances, excessive algal growths and low dissolved oxygen, which may also adversely affect water quality [*Pitt*, 2005], and thus pollutant removal rates may be lower during this period. Results from a study of wet detention ponds in North Carolina, showed that TSS removal can be correlated to surface area ratio. A pond surface area/ catchment ratio of 1% was required to achieve above 70% TSS removal. In order to provide 80% or better TSS removal, the surface area/ catchment ratios would need to increase to 2% or greater [*The Water Resources Research Institute*, 1989].

2.4.1 Particle size and distribution

As discussed previously in section 2.2.2, sediment particle size is extremely important in determining pond water quality, since it is one of the major factors which governs settleability. The Hjulstrom Curve (Figure 2.11) is a diagrammatic representation of how the phases of particle transport (erosion, transport and deposition) change with particle size and flow velocity. The curve itself was produced from empirical data "obtained for mono-disperse material on a bed of loose material of the same size of particles" [*Graf*, 1984] and defines the threshold flow velocities required to initiate particle motion [*Mayhew*, 1997]. As can be seen from the curve, larger particles are more likely to settle out of suspension than smaller particles. One exception to this concerns clay particles, which are extremely cohesive and may aggregate (floc) together to form larger, denser particles which settle out more quickly than individual clay particles.

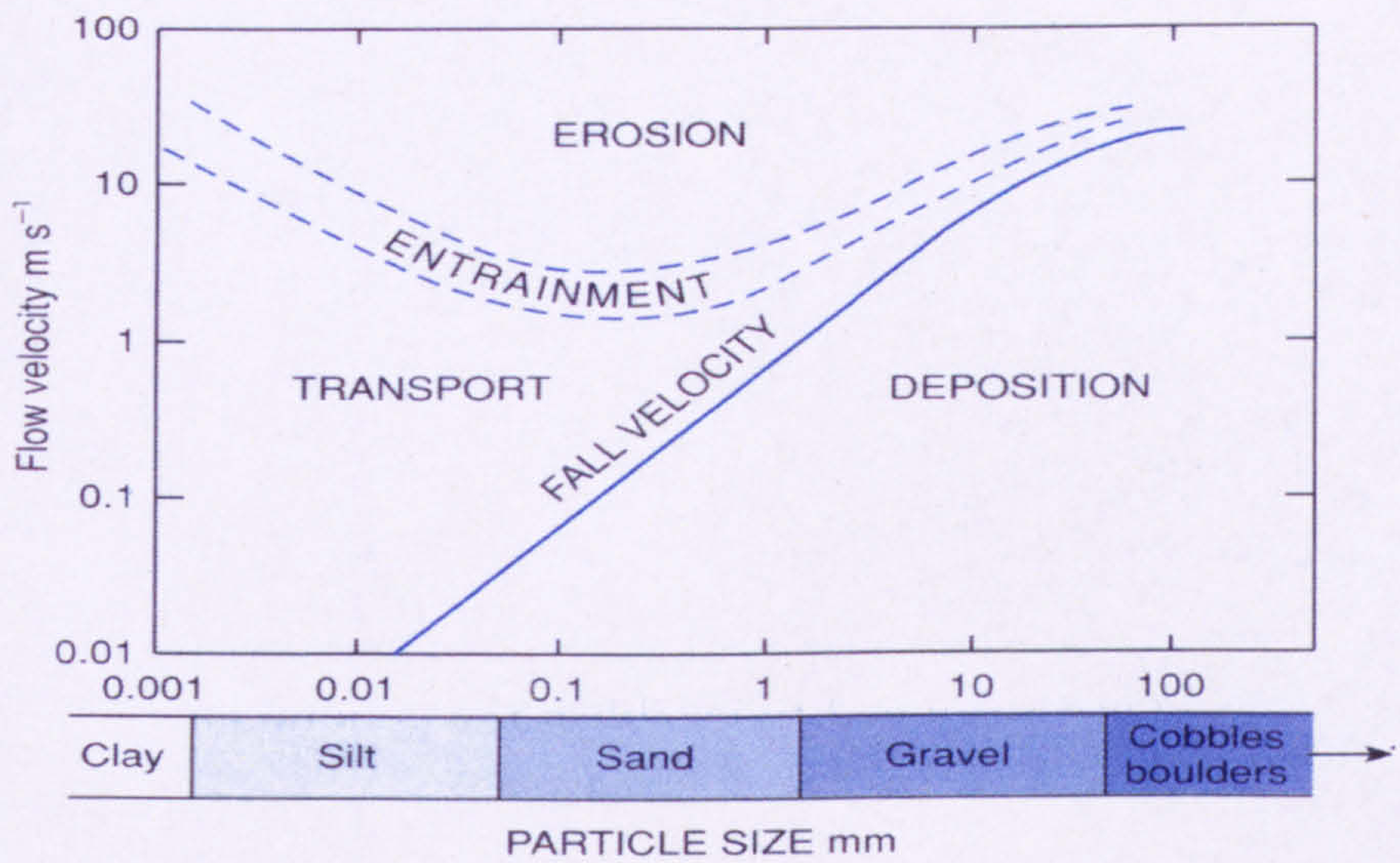


Figure 2.11: The Hjulstrom Curve, showing the phases of particle motion for different size fractions and flow rates.

2.4.2 Settling and Residence Time

By design, a proportion of the sediments fcarried into SUDS basins settle out. Literature discussing patterns of settling of sediments suggests that the process is highly variable, being largely dependant on particle size, density, and the nature of the flow [*Delleur*, 2001; *Krishnappan*, 2002]. However, it is generally accepted that the finer fraction takes longer to settle out of suspension in the water column [*Rushton*, 2001]. The processes of flocculation and disaggregation may affect the rate of settling within a pond since they cause changes in particle size and density [*Krishnappan*, 2002]. Flocculation is a mechanism whereby fine particles aggregate together to form larger particles that are more likely to settle out of suspension. It often occurs due to flow processes, such as advection, that promote contact between particles. In contrast, disaggregation is the process where flocs (the aggregated particles) break up. This process is often initiated in turbulent conditions within the pond and, since the resultant particles are often smaller and less dense, particle settling efficiency declines.

According to the [*EPA*, 1999] and [*Federal Highways Administration*, 2004] pond volume and depth exercise the greatest influence on particle settling. Both pond volume and depth determine the hydraulic residence time. The residence time of a parcel of water (and its sediment load) increases with increasing pond volume. The longer a particle of sediment stays in the water column, the more opportunity there is for particle settling, thus longer hydraulic residence times are favourable for water quality improvement in ponds [*EPA*, 1999]. While studies such as the National Urban Runoff Program (NURP) suggest that a large proportion of the polluting material settles out within 24 hours, [*Hartigan*, 1989] suggests an optimum hydraulic residence time of 14 days. This optimum Hydraulic Residence Time is provided by a permanent pool sized to contain 4 x Treatment Volume (Vt) as suggested in the [*CIRIA*, 2000] design guidance.

The settling out of particles causes the build-up of a layer of sediment on the base of a pond. Rates of sediment accumulation vary between ponds. This is due to differing sediment yield ratios between catchments and to differences in hydraulic conditions within individual ponds. A number of studies have shown sedimentation rates in basins draining urban areas ranging from 8-80mm per year [*Farm*, 2001; *Guo*, 1997; *Marsalek*, 1995; *Yousef et al.*, 1990]. However, acccording to EPA, typical sediment accumulation can range from 6-13mm per year in stormwater management basins, although this can be up to 100 times greater whenever construction activities are underway in the tributary watershed [*EPA*, 1999].

Although settlement of sediments is often advantageous from a water quality perspective, sediment accumulation adversely affects performance by reducing flood storage capacity and increasing overflow risk in the pond. Sediment accumulation also reduces treatment volume and shortens hydraulic residence times (the length of time that runoff, and thus suspended sediment, remains in the pond) reducing the effectiveness of contaminant removal [*Chabir et al.*, 2000; *Guo*, 1997]. A study by Guo (1997) showed that an annual sediment accumulation of 8mm in a stormwater management pond had the effect of reducing the flow attenuation capacity of the basin. During 18 years of operation, the basin storage capacity of the pond decreased from the equivalent of a 13-year to a 4-year storm, while other research has shown that pond storage may decline by as much as 13% over a ten year period [*The Water Resources Research Institute*, 1989].

In unlined ponds the settling out of sediments may also pose a potential threat to groundwater quality. Metals, unlike organic matter are not degraded in the environment and constitute an important class of persistent constituents due to the high risk of leaching of particulate-bound heavy metals into groundwater systems [*Ellis*, 2000; *Sansalone and Buchberger*, 1997b]. High concentrations of heavy metals have been found in soils served by infiltration systems [*Ellis*, 2000] and zinc, lead and copper have accumulated in the surface sediments of urban storm water detention basins in California at concentrations higher than the background concentrations [*Nightingale*, 1975]. However, it has been argued that this risk in wet detention ponds is negligible if they are dredged at 25-year intervals [*Heal*, 1999; *Yousef and Yu*, 1992].

2.4.3 Particle Resuspension

Resuspension of particulates has an undesirable effect on water quality in stormwater basins, since this increases the potential for particles to pass, untreated, out of the system into the receiving watercourse. Hydraulic conditions within the pond largely control the settling patterns of sediments and the likelihood of resuspension, for example turbulent flows, bed shear stress and disturbances caused by aquatic life within the pond may hinder particle deposition or may initiate particle resuspension and entrainment of previously settled sediments, although these are difficult to quantify [*Chabir et al.*, 2000; *Krishnappan*, 2002].

Suspended sediment also has a negative effect on dissolved oxygen concentrations in the pond, due to the decomposition (by oxidation) of organic matter contained within the sediment. The sediment oxygen demand (SOD) depletes the available dissolved oxygen in a pond, which can compromise the viability of aquatic species. Suspended sediments can consume as much as 4.96 mg^{-1} of oxygen per day [*Chabir*, 2000]. Limited evidence shows that bed disturbances such as storm-induced scour or resuspension may elevate SOD levels to the range of 240-1500g m^{-2} day^{-1} [*Ellis and Hvitved-Jacobsen*, 1996]. The effect of this process has not been quantified.

Summary

Recent studies have shown that both retention ponds and detention basins may contribute to water quality improvement. Pollutant removal efficiency varies greatly in SUDS basins and is highly dependant upon the physical processes which determine the availability of sediments, their ability to adsorb pollutants and on the three main factors that influence pond performance; sediment particle size and distribution, settling and residence time and particle resuspension. Modelling the processes which define sediment attenuation in SUDS ponds captures a significant influence on water quality enhancement. Chapters 5 and 6 of this thesis use a numerical model to investigate the factors that affect water quality performance, paying particular attention to the role of sediment. The simulations aim to explore the optimum pond conditions for water quality enhancement.

2.5 Current Design Standards

SUDS have been applied in many countries world wide, and consequently, design guidance varies to suit different environmental conditions and institutional and constitutional practices. Various studies have shown that countries have different preferences for the type of SUDS (or Best Management Practices, BMPs, as they are referred to in most of Europe) used. The section below briefly outlines the state of practice elsewhere in the world and provides a summary table of design standards for different countries.

2.5.1 Overview of Non UK Design Standards

Globally there are a range of terms which describe sustainable urban drainage, as its known in the UK. In the United States it is referred to as stormwater best management practice (BMP). In Australia, these systems are known as stormwater best practices but are utilised within the larger framework of Water Sensitive Urban Design (WSUD) [*Greenway*, 2003]. Furthermore, in different countries there are many different terms employed to describe similar systems. For example, the system known in the UK as a retention pond can be referred to as a detention pond, an extended detention pond or an extended wet detention pond in other countries.

Even within the U.S. regulations differ from the local, state and federal governments. The environment, in terms of climate, topography and geology, changes vastly from west to east across the U.S., and the site specific nature of stormwater management BMPs is well-documented. Current US best management practice uses retention ponds for the dual purpose of flood control and pollutant removal, since this method has been shown to be both reliable and effective [*France*, 2002]. However, despite designers in the US having many years experience in stormwater management practices, it has become clear that although there are well defined and reliable methods for designing flood control basins, methods for designing basins for water quality treatment are complex and in comparison, are poorly defined [*Wang*, 1996]. In developing basins to perform both functions, designers need not only to calculate storm water storage, but must take into consideration how changing pond conditions will impact aquatic flora and fauna and established ecological systems.

The use of stormwater management BMP's in Northern Europe has become fairly commonplace in recent years particularly in France, Germany and Scandinavia. They are used much less frequently, however, in countries such as Spain, Italy, Portugal and Greece [*Daywater*, 2000]. In France the main driver for the acceptance of BMPs has been the significant increase in flooding over the last decade, and the focus of stormwater management has been to mitigate increased urban peak flows. In France, retention systems, permeable paving solutions and underground reservoirs are widely employed to manage stormwater flows [*Daligault et al.*, 2001; *Daywater*, 2000]. In Sweden and Denmark, stormwater management practices have long been accepted and this is primarily due to a constitutional pro-active approach to environmental protection. Retention ponds are used frequently in these colder climates to help deal with stormwater runoff as well as the annual spring melt-waters, with particular concern for water quality protection [*Daywater*, 2000; *Persson et al.*, 1999; *Pettersson*, 1999a].

In both the USA and in Australia it is very apparent that the design guidance which is provided by the state comes from years of extensive monitoring and design projects alongside the input of academic research. This pool of information is assimilated over a number of years and provided freely in the form of BMP design manuals, the calculations and design criteria of which are specific to the climate/hydrology experienced in that state. It is well acknowledged that BMPs are site-specific and need to be designed for particular hydrological regimes. For example, the Stormwater Industry Association, Australia, has developed a design guide (Water Sensitive Urban Design: Basic Procedures for Stormwater Source Control – A Manual of Australian Practice) which provides design guidance for every climatic region in Australia [*Stormwater Industry Association*, 2003] and which is available free of charge to those who wish to consult it.

2.5.2 U.K Design Standards

The design of SUDS in the UK is somewhat different to the design guidance in other parts of the world. This is due primarily to sustainable stormwater management practices being in their infancy, but also in part to the institutional and constitutional differences between Scotland, Northern Ireland, England and Wales.

In Scotland the EC Water Framework Directive has been transposed into the Water Environment and Water Services (Scotland) Act 2003. This act establishes a 'planning system for the water environment with SEPA as the lead authority working alongside the public, private and voluntary sectors' [*Scottish Executive*, 2001]. In terms of SUDS, the act clarifies the responsibilities of each organisation and contains provisions which amend the Sewerage (Scotland) Act 1968 so that public SUDS are defined and have the same status as sewers. In practical terms, this means that the operation and maintenance of all public SUDS will be adopted by Scottish Water. It was also intended that The Scottish Executive would approve construction standards for SUDS by summer 2007 to provide the regulations that Scottish Water would apply to all SUDS, and by which it would be decided whether SUDS are suitable for adoption by Scottish Water. Scottish Water will not be responsible for private SUDS, or SUDS that convey only road drainage [*SEPA*, 2003]. These guidelines are currently at the consultation stage and are discussed in more detail in Chapter 7.

Until the Scottish Executive guidance is available, SEPA provide guidance on a number of SUDS related issues. This includes guidance for SUDS implementation on brownfield sites and how to conduct a drainage impact assessment (specific to Aberdeenshire as yet). Other advice including booklets and newsletters and general guidance on pollution prevention, for example PPG01, can be found on the SEPA website. Further guidance on the planning of

SUDS is given from the Scottish Executive through Planning Advice Note, (PAN) 61, [*Scottish Executive*, 2001].

Up until now UK-wide guidance has been provided by CIRIA. In their manual, the section covering design standards suggests that an economic analysis should be performed to asses the benefit of the pond, basin or reservoir based on the cost of damage prevented against the cost of construction when determining the 'design frequency' for a system.

Although the Water Framework Directive has also been transposed into law in England and Wales, SUDS have not been specifically addressed within the new laws. Policies specifically referring to SUDS are still being refined in England and Wales, however, the EA clearly state that they have two key policy objectives in stormwater management:

- **Primary objective**: to establish Sustainable Drainage Systems (SUDS) as normal drainage practice where appropriate for all new developments in England and Wales.
- **Secondary objective**: to retrofit SUDS on those existing surface water drainage systems which have an adverse effect on the environment.

Furthermore the EA organised a consultation with a wide range of stakeholders in the summer of 2003, with the aim of providing a Framework for Sustainable Urban Drainage Systems. The Framework itself was intended to outline a set of standards, or Code of Practice, for all of the relevant stakeholders. The response from the consultation document highlighted the primary concerns of stakeholder organisations as being those relating to maintenance, ownership and adoption of SUDS. In the absence of national policy to clarify the issues relating to the maintenance and adoption of SUDS, the National SUDS working group has produced an Interim Code of Practice for SUDS based on the proposed Framework and the response from the consultation exercise [*EA*, 2004].

It is only recently that information regarding the planning and construction of SUDS has been provided by the Environment Agencies in Scotland, England and Wales. To date most SUDS have been designed based on the CIRIA manuals; *Design of Flood Storage Reservoirs* [*CIRIA*, 1993] and *Sustainable Urban Drainage Systems Design Manual for Scotland and Northern Ireland* [*CIRIA*, 2000]. Furthermore, the EA website [*EA*, 2004] continues to cite CIRIA *Design Manual for England and Wales* and *Best Practice Manual* as reference material for technical guidance on SUDS, alongside the government Policy Guidance Note 25, *Development and Flood Risk.* These CIRIA publications were written by a collaborative team which included members from the Scottish Executive, SEPA, the Water Authorities, Local Authorities, Transport Authority and a handful of academic partners. Despite the wide collaboration, relevant case studies in the UK at the time of printing would have been few in number and the experience on which to draw from would have been reliant on case studies from other parts of the world (thus neglecting the site specific nature of SUDS) or from projects still undergoing the 'establishing' phases of their design life. The CIRIA manuals provided the first set guidance on SUDS systems and as such led the way in stormwater best management practice in its early stages in Scotland. However with the benefit of hindsight and experience, the manual has been criticised for its simplicity and lack of technical/scientific content. The need for continual research and further improvement is recognised in CIRIA's commitment to updating its publications.

In the sections that follow the current guidelines are discussed in more detail and compared to international standards in the following sections. It has been announced, however, that new guidelines are due to be published by CIRIA in 2007.

2.5.3 Factors Affecting Stormwater Management Basin Design

Table 2.3 has been compiled from a search of all the SUDS design guidance available in other countries to enable a comparison to be made between international and U.K guidance. The information presented in the table is not exhaustive, but aims to be representative of the guidance being used in countries that presently endorse the use of SUDS and their design methodology.

The table shows the design guidance for stormwater basins from a number of countries alongside current guidance in the UK. It is structured into 4 sub-tables: Table a) lists the design guidance for sizing of stormwater management basins for flow attenuation, Table b) provides design guidance on sizing and design for water quality enhancement, Table c) presents design guidance on pond hydraulics, and the guidance in Table d) refers to health and safety issues. As can be seen from table 2.3 a), b), c) and d), there are many design features that affect optimum pond performance. The main influences on stormwater management basin design are discussed below.

Sizing for Flow Attenuation

Since flow attenuation is one of the primary functions of a stormwater management basin, much of the literature discusses how to design basins for efficient flow control. The design guidance for flow control, shown in Table 2.3a) focuses largely on sizing the pond, or effectively how to build the pond to provide adequate temporary storage.

Table 2.3 (a) Designing for Flow Attenuation

Design Standard	Function	Reference	Country
Sizing for Flow Attenuation			
Plan area should be 10-25% of the total basin area	To provide adequate flood storage volume	CIRIA (2000)	U.K
Should attenuate the 1 in 25yr and 1 in 100yr flood event, but higher standard imposed where potential loss is higher	To protect from both large, infrequent storm events and smaller more common events	CIRIA (1993)	U.K
Capture volume should be sized to contain 85% of the annual runoff volume	To provide adequate flood storage volume	California Stormwater BMP handbook (2002)	USA
Must be sized to provide detention of the 1,2,5,10 and 100yr event	To ensure protection from frequent small storms and large rare events	City of Knoxville, Tennessee (2003)	USA
The surface area of basins should be at least 1% of the contributing catchment area	To provide adequate storage for catchment runoff	DCR (2002)	USA
Post development flows should match the volume, shape, and peak instantaneous rates of pre-development flows for the 6month/24 hour precipitation events	To ensure SUDS are ameliorating the effects of urbanisation on catchment flows	Mapleridge BMP Guidelines (2001)	Canada
Volumes from the post-development 6month/24hour events on impervious areas should not be discharged but should be infiltrated to the ground	To ensure SUDS are working towards reinstating more natural catchment flows	Mapleridge BMP Guidelines (2001)	Canada

Specific procedures for sizing stormwater management basins in the UK are discussed in full in section 2.3.1. In terms of protecting against floods of a particular magnitude, [*CIRIA*, 1993] advise that stormwater management basins be designed to protect against the 25 year and the 100 year event. If the consequences of failure are serious (e.g. substantial loss of life), the magnitude will be increased (e.g. 1 in 500 year return period).

Guidance outwith the UK varies considerably for the sizing of stormwater management basins, even from state to state in the USA. According to [*DCR*, 2002], the surface area of basins (and extended detention basins) should be at least 1% of the contributing catchment area, while the California Stormwater BMP Handbook suggests that the capture volume should be 85% of the annual runoff volume to ensure adequate storage for flood protection. In Canada, the design for flood protection is approached in a different manner with [*Mappleridge BMP Guidelines*, 2001] guidelines suggesting that volumes from the post-development 6-month/24-hour events on impervious areas should not be discharged, but should be infiltrated to the ground and the post development flows should match the volume, shape and peak instantaneous rates of pre-development flows for the 6-month/24-hour, 1 in 2yr/24-hour and 1 in 5 yr/24-hour precipitation events.

All US guidance emphasises the importance of sizing for small and large events (typically including the 1, 2, 5, 10, 25 and 100 yr events) to enable basins to perform both functions of water quality enhancement and flow attenuation. Perhaps one of the major flaws in UK guidance is that design guidance for basin sizing is based on, and on numerous occasions refers to, a previous manual which is specifically written to cater only for flood storage reservoirs [*CIRIA*, 1993]. In order for basins to perform both functions adequately, they must be designed to cater for both large and small rainfall events. In terms of designing a pond for flood abatement, it is the large, relatively infrequent events which are significant, and the design priority would be in ensuring enough adequate storage provision during these rare events. However in designing the pond to provide water quality enhancement, it is small, frequent events that are significant, since it is these events which are responsible for the mobilisation and transport of urban surface pollutants [*Campbell*, 2004]. This clear division of design priorities creates a critical conflict for stormwater management basin design.

Design for Water Quality

In the UK, only retention ponds are designed to improve water quality. Table 2.3b) provides a summary of the guidance for designing stormwater management ponds for good water quality enhancement. In much of the guidance material there are several different ways to achieve efficient treatment of polluted water including sizing the pond, pre-treatment and inclusion of aquatic vegetation and forebays. However, all of these different methods of achieving improved treatment are aimed at enhancing sediment (and in some cases solute) removal in the pond.

Table 2.3 (b) Designing for Water Quality

Design Standard	Function	Reference	Country
Sizing for Water Quality			
Basins should be 1% of the contributing catchment area	Provides 80-90% of solids removal for a 1 in 1 yr storm	CIRIA (1993)	UK
Permanent pool volume should be 4 times the design treatment volume	To provide adequate residence times (14-21 days)	CIRIA (1993)	UK
Permanent pool volume must have a minimum residence time of 14 days	To allow adequate chemical and biological treatment	City of Knoxville, Tennessee (2003)	USA
Storage volume should be 150-250m^3 per impervious hectare	Provides 50-60% removal of soluble material	CIRIA (1993)	UK
Ratio of pond surface area to contributing impervious area should be 250m^2/ha	Optimal ratio for pollutant removal, above which pollutant removal is negligible	Pettersson *et al.*, (1999)	Sweden
Water Quality Treatment Standards			
The first flush must be captured, detained and released over a 24 hour period	To meet water quality objectives (when first flush has a min volume of 45,000 cubic feet)	City of Knoxville, Tennessee (2003)	USA
Detention time should be between 24 and 40 hours (taking infiltration rate into consideration)	To allow adequate sediment settling and thus pollutant removal	France, (2002)	USA
Aim to collect and treat the first flush of smaller storms		Mapleridge Stormwater BMP Handbook (2001)	Canada
Collect and treat the volume of the 24 hour precipitation event equalling 90% of the rainfall from impervious areas		Mapleridge Stormwater BMP Handbook (2001)	Canada
Nutrient loads (P and N) must be reduced by 45% and phytoplankton and macro algae counts taken.	Reduces the possibility of algal blooms	Melbourne Water (2000)	Australia
Flow depth should be no less than 1 ft for an annual mean storm event	To ensure adequate treatment volume	France (2000)	USA
Stormwater should undergo pre-treatment before discharge to pond	Removal of oil and grit prevent endangering ecological communities and clogging of pipes	France (2002)	USA
Minimise 'clean water', such as roof water through the pond	Maximises treatment of 'dirty water', such as road runoff	France (2002)	USA

Water Quality Design Features			
Basin should be designed as a two-stage storage facility with a sediment forebay and a main pool	To isolate gross sediments and simplify water quality treatment	California Stormwater BMP Handbook (2002)	USA
Provide a sediment forebay at the inlet	To enhance sedimentation	France (2002)	USA
Outlet structures should be designed for low outflows during low pond depths	To maximise particle retention	Pitt (2002)	Canada
Preferable water depth is 1.5-2m	Promotes a productive biological community establishment which aids treatment	Melbourne Water (2000)	Australia
A geotechnical investigation must be conducted	To establish groundwater depth and quality including nitrates, dissolved metals and organic content	Melbourne Water (2000)	Australia
Draw down times should be 48 hours	longer draw down encourages breeding of mosquito and shorter periods do not allow settling	California Stormwater BMP Handbook (2002)	USA
Planting of pollutant-tolerant vegetation	To enhance in pond water quality and provide amenity	France (2002)	USA
Planting of shallow fringing vegetation	Enhances nutrient, metal and hydrocarbon uptake. Vegetation will shade surface water to mitigate warming	CIRIA (1993)	UK

According to [*German*, 2003], pond size is a critical design consideration. In stormwater management pond in Sweden, size is often measured using a ratio of pond surface area to the contributing impervious area, known as the SAR ratio. Research shows that pond removal efficiency is related to SAR and is optimal when the surface area of a pond is around 2% of the contributing impervious catchment area [*Novotny*, 1994]. Further, [*Pettersson*, 1999b] showed that pollutant removal efficiency increases up to a specific value of pond surface/impervious area of $250m^2/ha$, above which the removal efficiency of the pond only increases marginally with increased pond area. Guidance for UK stormwater management basins recommends that the pond be sized to be 1% of the contributing catchment area, which should correspond to the removal of 80% of solids for the 1-year event [*CIRIA*, 1993]. This manual also advises that the storage volume of the pond should be $150\text{-}250m^3$ per impervious catchment hectare in order to achieve a 50-60% removal of soluble material. CIRIA (1993) provide further guidelines on the sizing of a basin for water quality treatment recommending that the permanent pool volume should be sized to contain four times the design treatment volume. The treatment volume is defined as the volume of surface runoff containing the most polluted portion of the flow from a rainfall event, and it should be retained in the pond for 14-21 days [*CIRIA*, 1993].

Pond Hydraulics (Table 2.3c)

Pollutant removal capacity of a pond is highly dependent on pond hydraulics. Factors that increase the hydraulic performance are principally related to basin shape including length to width ratio, location of inlets and outlets, topography and the presence of islands and baffles, amongst others (Persson, 2000). Furthermore the water quality performance of a pond is influenced by hydraulic retention time and the provision of adequate mixing in the pond to prevent short-circuiting (CIRIA, 1993).

Table 2.4(c) Designing for Pond Hydraulics

Design Standard	Function	Reference	Country
Pond Hydraulics			
Water Depth			
Depths are limited to 3m	Prevents thermal stratification	Lawrence et al., (2001)	U.K
At least 50-70% of depth should be no less than 1-1.5m	Encourage oxygenation	CIRIA (1993)	U.K
Water depth should not exceed 8ft		California Stormwater BMP Handbook (2002)	USA
A minimum of 3ft (preferably 6ft) of permanent standing water	Reduces scouring which can decrease light penetration	Pitt (2005)	Canada
Depths limited to 3m	Allows adequate light penetration and reduces thermal stratification	Melbourne Water (2000)	Australia
Length to Width Ratio			
Length to width ratio should be at least 3:1	Minimises short circuiting	CIRIA (2000)	U.K
Minimum length to width ratio of 1:3-1:4 required	Long, narrow ponds do not promote mixing of the peak flows with pond water	CIRIA (1993)	U.K
Length to width ratio should be 3:1	Prevents short circuiting and improves sediment removal	France (2002)	USA
Length to width ratio of 2:1	Improves hydraulic efficiency, encourages plug flow and reduces short circuiting	DCR (2002), Knight (1987)	USA
Length to width ratio of 3-4:1 required	Maximises pollutant removal with increasing loading rate	Reed et al., (1995)	USA
Length should be 3-5 times the width	Maximises detention efficiency	Pitt (2005)	Canada
Length to width ratio should be minimum of 1.5-1	To reduce short circuiting	California Stormwater BMP Handbook (2002)	USA
Length to width ratio should be at least 2:1 (preferably 3:1)	To reduce short circuiting	City of Knoxville, Tennessee BMP manual (2003)	USA
Length to width ratios should be 5:1	produces maximum pond efficiency (reduces short circuiting)	Hitman (1976), Pitt (2002)	USA, Canada
Inlets and Outlet Design			
Maximise the distance between inlets and outlets	Improves hydraulic efficiency and reduces short circuiting	CIRIA, (1993)	UK

Maximise distance between inlet and outlets	Prevents short circuiting and improves sediment removal	France (2003)	USA
Inlet design should incorporate a cascade or stepped section	Encourages mixing, oxygenation and siltation	CIRIA (1993)	UK
Inlets and outlets need to be widely spaced	Minimises short-circuiting	Pitt (2005)	Canada
Outlet device should be designed to have multiple weirs/orifices	To release all design storms (1, 2, 5, 10 and 100yr) at pre-development rates	City of Knoxville, Tennessee	USA
Outflow should have multi-level discharge points	To provide adequate detention time for storms <2yr event	California Stormwater BMP Handbook (2002)	USA
Outlet structure should drain pond down to the permanent pool level in 24 hours	To ensure adequate storage is provided for future storms	California Stormwater BMP Handbook (2002)	USA
V-notch weirs and multi-stage outlets are recommended for general use	To control both low and high flow	Pitt (2005)	Canada
The lowest opening in a multi-stage outlet should be at the permanent pool level	To provide both the desired water quality and flood control benefits	Pitt (2005)	Canada
Flow distribution berms should be installed at the inlet and outlet	To promote uniform flow distribution	France (2002)	USA

Water Depth

All of the guidance agrees that the minimum pond water depth should be 1m to ensure that there is enough water to encourage settling, good oxygenation and to discourage basin scouring which increases turbidity and reduces light penetration [*California BMP Stormwater Handbook*, 2002; *CIRIA*, 1993; *Melbourne Water*, 2000; *Pitt*, 2005]. Much of the guidance provides an optimal or a preferred water depth, suggesting that this should be between 1.5-2.5m. Ideally depths should not exceed 3m to prevent thermal stratification and to meet health and safety requirements.

Length to Width Ratio

According to the Water Pollution Control Federation [*Water Pollution Control Federation*, 1990], optimal hydraulic conditions (hydraulic efficiency) for water quality improvement occur under plug flow conditions, since this type of flow is typically characterised by a uniform velocity profile, where parcels of water move parallel to the basin sides with very little vertical dispersion. This type of flow is encouraged by ponds which have a long, narrow geometry, and thus basin shape plays a very critical role in the hydraulic characteristics of a pond. Furthermore, research has shown that maximising length to width ratios reduces short-circuiting, (short flow paths which allow water to enter and leave the basin quickly with very little or no treatment) since there are less likely to be dead zones along the flow path from the inlet to the outlet [*DCR*, 2002; *Persson*, 2000].

The guidance on hydraulic performance and flow characteristics is somewhat contradictory. Different authors suggest different length to width ratios for optimum hydraulic efficiency, defined by Wong and Somes (1995) as how well incoming water distributes within the pond. Optimum length to width ratios suggested by various researchers range from 2:1 – 10:1 [*France*, 2002; *Knight*, 1987; *Persson*, 2000; *Pitt*, 2005; *Reed*, 1995; *Water Pollution Control Federation*, 1990]. While most of the literature advises that these high length to width ratios are beneficial since they encourage plug flow (and therefore uniform velocity profiles which promotes channelised flow), some UK guidance suggests that plug flow is not desirable since incoming pollutants travel down the length of the pond relatively unhindered, increasing the risk of short-circuiting and that this problem can be reduced by limiting length to width ratios [*CIRIA*, 1993]. To further confuse the issue, [*Kadlec and Knight*, 1996] suggest that the occurrence of wind induced mixing renders pond hydraulic behaviour completely independent of basin shape. Despite this, it is generally accepted that long, narrow ponds improve hydraulic performance since research has shown that such geometries improve pollutant removal rates for Total Suspended Sediment (TSS), Total Nitrogen (TN) and Biological Oxygen Demand (BOD) [*Knight*, 1987]. They also increase the effective volume, increase the mean detention time [*Mathews*, 1997] and perhaps most importantly reduce short-circuiting [*Persson*, 2000].

Inlets and Outlets

Good hydraulic conditions in a pond can also be optimised by maximising the distance between inlets and outlets to ensure longer flow paths for pollutants [*CIRIA*, 1993; *France*, 2002; *Pitt*, 2005]. This will promote settling by maximising retention times and reducing short circuiting. Several researchers in the U.S. and Canada suggest that outlets should be designed with multiple outflow controls or multi-level outflows [*France*, 2002; *Pitt*, 2005; *Tennessee BMP Guidelines*, 2003]. Outlets are designed in this way to ensure that ponds can attenuate a range of event magnitudes from the smallest (<2 yr-event) to the largest (>100 yr-event) as well as being able to provide adequate water quality detention times.

Pond Side Slopes

In the UK the pond side slopes are a major consideration, and gradients are typically no steeper than 1 in 4 (1 vertical unit to 4 horizontal units) [*CIRIA*, 1993]. However, this is usually to ensure that ponds meet the UK's strict health and safety requirements, rather than for improved hydraulic performance. In other countries, side slopes may range from 1 in 4 to 1 in 10 (i.e. shallower). Again this may be due to health and safety obligations, or due to engineering and maintenance requirements which need to ensure slope stability and access for weeding and grass cutting etc., [*Chambers*, 1980; *Pitt*, 2005; *Scheuler*, 1987].

Pond design features

The ability of the pond to remove sediments can be improved further with the addition of baffles and islands, which can prevent short-circuiting by reducing dead zones and encouraging preferred flow patterns [*Persson*, 2000; *Persson et al.*, 1999]. Consequently, the quiescent conditions created within the pond promote particle settling and prevent resuspension. Research by [*Persson*, 2000] suggests that placing an island at the inlet encourages particle settling since the island creates a barrier to flow, thus reducing inflow velocities. Advice from the [*CIRIA*, 1993] design manual is consistent with this, recommending that flow velocities at the inlet should be restricted to 0.3-0.5 m/s.

Health and Safety

As well as being designed to provide good flow attenuation and efficient treatment of polluted water, ponds must also be designed to meet health and safety requirements, particularly since many new SUDS developments are within residential and commercial areas.

Table 2.3(d) Designing for Health and Safety

Design Standard	Function	Reference	Country
Health and Safety			
Open water in ponds should occupy 50-75% of the permanent pond surface		CIRIA (2000)	UK
Water depth in the permanent pool should be 1-2m (max 3m)	Health and safety	CIRIA (2000)	UK
Pond slopes limited to 1 in 4	Health and safety	CIRIA (2000)	UK
Minimum slope of 3:1	minimise public hazard	France (2000)	USA
Safety level bench (5-10ft) should be installed at depth of 3ft	Minimise public hazzard	France (2002)	USA
Pond depth should vary and a safety bench	Health and safety	Melbourne Water (2000)	Australia

should be incorporated

Table 2.3d) highlights the need to provide shallow ponds with gentle side slopes to reduce the risk that open water provides to the public. Many of the design manuals also encourage the incorporation of safety features such as safety benches.

2.6 Conflicting issues for Design of Flow and Water Quality

A basin's flow attenuation effectiveness is determined by a number of factors, namely, basin storage capacity, hydrologic regime, draw-down time and by the outlet device and its configuration.

2.6.1 Basin Storage Capacity and Draw-Down Time

Retention ponds and detention basins are designed specifically to reduce peak runoff rates to pre-development levels. Moreover, they are designed to attenuate a single storm event size, rather than a sequence of storm events. Different combinations of rainfall magnitude and duration determine the intensity of storms [*Jones*, 1997]. How a basin performs, in terms of its ability to provide adequate storage capacity, ultimately depends on its response to the varying quantity and timing of runoff. Furthermore, how a basin performs over its design life will be determined by how it responds to continual sequences of varying inflow. Clearly creating a large temporary storage capacity is desirable for flood risk reduction. Detention basins ar e designed to remain dry in the inter-event period by encouraging complete emptying between storms. According to [*Guo*, 2002], detention basins provide optimum flood risk reduction since they drain completely in the inter-event period and always provide 100% temporary storage capacity. Retention ponds and wetlands are designed to contain a permanent body of water. Here, the emphasis has traditionally been on water quality treatment through sedimentation, which requires long settling times. Such basins do not provide optimum flood risk reduction, since the permanent pool of water reduces temporary storage capacity.

In terms of flow attenuation performance, the conflict of interest between designing the pond to drain slowly in order to detain stormwater to promote good flow attenuation and designing the pond to drain quickly to enable adequate storage for future storms is a key issue in stormwater basin design. According to Guo (2002) there are two important phases in basins designed to reduce flood risk; the waiting period between storm events and the draining period after an event. In order to reduce overflow risk, defined as the probability of having a rainfall event that produces runoff volume greater than the available storage capacity in the basin, draining of the basin quickly in preparation for the next storm event is essential [*Guo*, 2002].

There is a further conflict of interest between design for optimum flow attenuation and design for optimum water quality enhancement. This issue arises because in order to improve water quality, a long residence time for water is required to encourage the settling of sediments. However, as discussed previously, the provision of adequate storage area for future storms (referred to later the thesis as the Temporary Storage Volume, TSV) is critical for flow attenuation. If the availability of this storage area is compromised by having a large body of water stored for water treatment, a trade off between the two pond functions arises. The solution to this problem is the design of a permanent pool which should be available all year round for water quality enhancement, while the rest of the volume above the pool can be used for temporary storage of stormwater. As is examined in later chapters, the TSV plays a key role in both flow attenuation and water quality treatment, and is in fact the essence of optimum pond design.

The hydrologic regime is an important consideration in stormwater management basin performance since the length of the antecedent period determines the time the basin or pond has to drain down before the occurrence of the next storm. The antecedent period also determines catchment wetness and thus short antecedent periods are not only likely to result in pond failure and overtopping, but may also coincide with catchment conditions with a high probability of flooding. An equally important influence on pond and basin performance is the storm sequence and magnitude. Stormwater management basins must be designed to capture large storms, since these are important for flood control. However, ponds must also be able to drain smaller storms since it is these storms that are critical in terms of water quality [*Campbell*, 2004; *New Jersey Department of Watershed Management*, 2003] herein a further conflict in retention pond design arises. This is discussed in further detail in Chapter 6.

Finally, flow attenuation effectiveness is also influenced by the outlet device. For example, when water levels reach the crest of a v-notch weir, the weir angle plays an important role in determining the rate of discharge. While larger weir angles reduce the flow attenuation performance of the pond by allowing too much water to discharge, caution must be exercised in using smaller weir angles, which, despite improving performance, may lead to an increased risk of overtopping. Ultimately the outlet device must be designed appropriately to suit the local hydrologic regime, enabling the pond to drain down in sufficient time in accordance with typical antecedent period lengths for the region. Ponds can be designed with different outlet configurations consisting of single weirs, multiple weirs and a combination of weirs and pipes and/or hydrobrakes. Hydrobrakes and pipes are very efficient in throttling the flow, where as the weir is more efficient in rapid drainage of the pond, and for this reason is often used to prevent flooding of the surrounding area. A study that compared two detention basins with different outflow devices (receiving only highway runoff from roads serving the surrounding commercial and residential areas) showed that a better reduction in peak flow was achieved when a vortex outlet (hydrobrake) control was used, as opposed to a submerged pipe outlet [*Pratt*, 2001; *Sniffer*, 2001]. Chapter 4 examines the effect of outlet device configuration on pond performance, however the performance of hydrobrakes is not considered.

In summary, a number of conflicts arise in designing stormwater management basins to provide good flow attenuation and water quality enhancement. A balance must be achieved between a number of factors. Among the most important are the compromises between storage capacity and draw down time, between designing for large and small storms, and in choosing appropriate outlet devices. Furthermore, a retention pond must be designed to achieve all of these compromises as well as being designed with due consideration of treatment volumes, catchment land-use characteristics and climatology whilst also meeting the standards set for health and safety, pond aesthetics and providing wildlife habitat.

Although it is well known that all of these factors influence the performance of retention ponds, currently the effect of each one has not been quantified. Retention ponds have only recently been introduced to the UK and thus data on hydrological performance is sparse and short. It is unclear how retention ponds built using current design guidance will perform through-out their design life and how they might respond to changes in climate.

Furthermore, there is a clear lack of precise design guidance for developers of retention ponds in the UK. The thesis focuses on the issues raised in this chapter and, by investigating the influence of flow attenuation and water quality enhancement on the performance of ponds and achieve the following research aims:

1. Quantify the flow attenuation performance of retention ponds
2. Investigate pond design for optimum pond flow attenuation performance
3. Assess how performance might change under changing climatic conditions.
4. Quantify the water quality performance of retention ponds and investigate the potential water quality performance of detention basins
5. Investigate pond design for optimum water quality performance
6. Assess how ponds can be designed to achieve both flow and water quality targets and identifying the obstacles that might prevent it
7. Propose improvements to current design guidelines

3 Pond Model Development

Due to a lack of availability of a comprehensive data set (i.e. one that included inflow and outflow data over the same period of time for any one pond system), it was imperative that a model be used to enable a comparative investigation of pond design on performance. Due to the difficulties in obtaining data, the modelling work in the thesis focuses on the design of various hypothetical pond configurations. This chapter provides an introduction to the design of the mathematical model used to simulate pond flows in Chapter 4. The chapter commences with an overview of other modelling applications relevant to SUDS ponds. The model applied within this thesis for the simulation of pond flows is then introduced and verified. Finally a sensitivity analysis of model parameters is described. The modelling of sediment is discussed in full in Chapter 5.

3.1 Review of Available Pond Modelling Software

There are many commercial stormwater modelling packages available. The most well-known of these, SWMM, MIKE STORM, MOUSE, DRINS, and SWMHYMO, have been employed primarily in watershed and sewershed planning. Other less well-known packages such as Catchment Sim and ERWIN are also available, and are designed primarily for application at field-scale rather than catchment-wide planning.

The ready availability of such off-the-shelf models, the friendly graphical user interfaces and the excellent user support should make this type of package ideal for investigative modelling. However, for this project a model was required that could model both flow and water quality aspects of stormwater management basins, furthermore, at the outset of the project there was a very limited supply of monitoring data for retention ponds and their catchments. Part of the problem with such complex commercial models is their requirement for large volumes of catchment data. SWMM, for example, is a physically based model that has been employed for a range of projects spanning stormwater management to floodplain analysis. Although SWMM is capable of running both single-event and continuous simulations as well as performing flow and water quality modelling, the model requires a large volume of data about the catchment and its function. Since various forms of precipitation (including snowmelt) can be input to produce prediction of flows, stages and pollutant concentrations, the runoff module alone requires details such as impervious area and slope of the catchment, depression storage, Manning's roughness for pervious and impervious areas, and either the Horton or Green and Ampt infiltration parameters. Although such details add to the complexity, and therefore the potential of the model to reflect catchment behaviour accurately, very few of these details are easily accessible.

The requirement for a large number of detailed catchment data is common to many of the commercial stormwater models (MIKE STORM, MOUSE, Watershed Bounded Network Model (WBNM), HYDSIS, and ERWIN). Furthermore many of the models only simulate either flow or water quality (ILSAX, DRAINS, SWMHYMO, MOUSE Trap, WBNM, HYDSIS, ERWIN), and although some are able to simulate catchment hydrology, flow and sediment movement in sewer systems and pipes, most are unable to simulate SUDS retention ponds.

Many of the newer models are heavily supported by graphical and object-oriented techniques which allow graphical data describing the drainage system, to be entered via drawing tools or transfers from CAD, GIS and digital terrain modelling (DTM) programs and spreadsheets (DRAINS, MIKE STORM, ILSAX, CatchmentSim, HYDSIS), such tools can be expensive and require a high level of expertise to operate them, while many of the models (in some cases modules) are unable to perform simulations unless they are coupled with existing rainfall generators, hydrological routing models, or more complex models (MOUSEtrap, CatchmentSim, ERWIN). A table comparing the capabilities of the models introduced here can be found in Appendix B. According to the table, none of the packages mentioned have the ability to model BMPs (Best Management Practices) i.e. SUDS, however ERWIN, (which is not reviewed in the table) was designed specifically for modelling stormwater drainage systems, and is capable of simulating both detention basins and retention ponds. The approach used is a simple input-storage-output method, which does not take account of pond geometry.

Since such limited functionality was required of the models to simulate stormwater management ponds, it was decided that the best approach would be to develop a stand-alone model at Heriot-Watt University. This was designed and developed by Dr. Steve

Wallis and has the advantage that the code can be modified or edited to incorporate new features if required.

3.1.1 Review of Pond Modelling Research

Sustainable urban drainage systems have been modelled to analyse their performance efficiency, their response to varying input parameters and their limitations. This investigative modelling has occurred in all types of SUDS including filter trenches and soakaways, wetlands and basins and swales and porous paving [*Backstrom*, 2002; *Butler and Memon*, 1999; *Deletic et al.*, 2000; *Ellis et al.*, 1995; *Schluter and Jeffries*, 2001; *Somes et al.*, 1999], while other models have been used to aid the design of SUDS [*Blazejewski and Murat-Blazejewska*, 2003; *Guo*, 1999; *Guo*, 1997; *Heitz et al.*, 2000; *Persson et al.*, 1999]

In the search of modelling literature that was undertaken, few studies could be found that consider the dual role of stormwater ponds for flow attenuation and pollution control. Many of the early studies focus on stormwater detention basins since it was initially this type of basin that was used as a means of flow control in urban catchments. As concerns for the quality of water became more prominent, more research was conducted on wetlands and retention ponds. Much of this work was been carried out in the United States, Canada and Northern Europe. The review presented here covers modelling work conducted on retention ponds and detention basins but also includes research on wetland systems used for both stormwater and wastewater drainage. Although wetland systems are not considered later in the thesis, and the modelling reviewed here does not concern the biological or chemical role that wetlands play in water quality improvement, modelling of the hydrology and the hydraulics is considered since it is also applicable in many cases to retention ponds. There are many modelling studies that do investigate the contribution that vegetation in wetlands and ponds makes to water quality improvement. Several of these are described in [*Somes et al.*, 1999].

While there are numerous studies that model the role of SUDS in large scale catchments or are used as catchment planning tools [*Elliot*, 1998; *Perez-Pedini et al.*, 2005; *Trauth and Adams*, 2004; *Wang*, 2001; *Yeh and Labadie*, 1997] there are relatively few studies that model individual pond behaviour and performance. For example, [*Elliot*, 1998] used a simple mathematical model to determine the optimal location of stormwater BMPs (or in the UK, SUDs) in a catchment for water quality improvement. BMPs had to achieve targets for groundwater recharge, flood control and concentrations of Zn and Cu in sediment in the outflow. An optimisation approach was then used to determine the most cost-effective stormwater quality controls. Likewise, [*Papa and Adams*, 1997] proposed a methodology for optimising the pond geometry of a single stormwater retention pond (designed solely for water quality improvement) using analytical probabilistic models for regional stormwater management planning and analysis. This methodology was extended so that multiple catchments could be modelled each having a single pond (with potentially variable performance) upstream of its outlet. Due to the potentially high values of the urban areas where ponds are developed, it is advantageous to minimise pond size whilst still achieving water quality objectives. The model thus estimates the collective pond performances and determines whether they meet a specified pollution control level at a single discharge point. Similarly [*Zhen et al.*, 2004] present a stormwater management tool that determines the optimal location and design of BMPs (SUDS) in a catchment. It combines heuristic optimisation techniques, a catchment model (AnnAGNPS) and a BMP simulation module to determine the most cost-effective stormwater management options that meet pollution control standards. The optimum locations for BMPs selected by the model were the ten 'hot spots' in the catchment that had the highest sediment loadings. The optimum BMP type selected by the model was an extended detention pond and the detention time and the average pond depth were selected as the parameters for optimisation since earlier work considered these to be the most critical in terms of detention pond performance. Results indicated that when the effects of sediment resuspension in stormwater ponds are not considered the system is likely to be under-designed. The study also highlighted the importance of simulating the loss of pond storage volume and depth as a result of sediment accumulation in a pond.

Of the studies that are concerned with the modelling of individual ponds, many model small-scale internal processes mainly associated with water quality such as flocculation and advection, or Hydraulic Retention Time (HRT). There are only a few studies that model only the hydraulic processes in ponds. For example predictive modelling work carried out by [*Persson*, 2000] attempted to relate pond design to pond hydraulic performance. A two dimensional numerical model was used to compare the performance of thirteen hypothetical ponds with different geometries to determine the layout that provided the optimum hydraulic performance (or minimised short circuiting). Two Mike21 modules were used in this study, the hydrodynamic (HD) and the advection-dispersion (AD) module. Mike21 is a two dimensional model that assumes that a water body is vertically homogenous. According to [*Persson*, 2000], this approach is well suited to a pond with depths between 0.5-2.0m. The modules require numerous inputs including bathymetry inflow, inlet boundary concentration and dispersion co-efficient. Other inputs are required to describe the shear stresses in the momentum equations including turbulence and velocity gradients.

The pond geometries studied were based on a literature review of pond design which suggests that pond shape, length to width ratio, topography (islands) and the location of inlets and outlets is important in pond hydraulic performance. The study aimed to find the design which produced the least short circuiting, since the study emphasises the relationship between pond hydraulic performance (short circuiting) residence time and pond water quality. The results were not verified on existing ponds, however this was thought not to be necessary since the study was a comparison between hypothetical ponds. Simulation results indicate that length to width ratio, the location of inlets and outlets and the provision of a subsurface berm all have a major influence on pond hydraulic performance. Short circuiting decreased with longer length to width ratios and the presence of subsurface berms. Furthermore, results suggest that an island placed in front of the inlet improves pond hydraulic performance. The effect of pond geometry and layout on the hydraulic efficiency and the effect of this on pollutant removal is discussed further in [*Persson et al.*, 1999].

Other work relating to individual pond performance is concerned with the sizing of stormwater basins. The models are often used to determine the most efficient basin designs for a number of different conditions, e.g. different rainfall recurrence intervals and magnitude events, different detention periods and different basin geometries. [*Guo*, 1999] presents a technique for reliable estimation of average outflow from a detention basin by devising a relationship between duration of the design storm, time of concentration of the catchment and the average outflow. This method improves the reliability of volume based mathematical techniques for sizing of detention basins. As part of a larger project, [*Heitz et al.*, 2000] developed a series of design curves from an in-depth analysis of local rainfall records and used them to enable the sizing of stormwater ponds. It is assumed that the detention basin is empty at the start of each storm event, thus the emptying time of the pond must be equal to or less than the period between storms. Other studies that consider the sizing of wetlands and ponds for water quality performance can be found in [*Wong*, 1995]

Most of the recent modelling work concerns improving pond water quality performance, reflecting a shift in political drivers away from the use of stormwater detention solely as a means of flood protection towards water pollution prevention. A number of studies have been conducted to determine the retention time of stormwater wetlands and ponds. Much of the initial modelling work was carried out on wastewater wetlands [*Kadlec and Knight*, 1996]. However such modelling approaches used steady state analyses which, according to [*Walker*, 1998]are inadequate for simulating the transient characteristics of stormwater inflows. [*Kadlec*, 1994] conducted tracer studies in a wetland to determine hydraulic parameters such as the Mannings number and dispersion coefficients to enable the predominant flow patterns that develop during stormwater inflow to be predicted. The study showed that rather than flow in the wetland being predominated by plug flow, the flow type was a composite of plug flow and well-mixed flow types. Furthermore the estimated detention time was 50% larger than the mean tracer detention time. The results from the study provided the basis for the development of three numerical 1-dimensional models; a plug flow model, tanks in series, and a parallel network of tanks. All three models were able to adequately simulate outlet tracer concentration curves, however, the parallel network model provided a better fit to internal wetland measurements.

[*Walker*, 1998] developed a mathematical model to determine the residence time in stormwater ponds and wetlands. The estimate of residence time was used to quantify improvements in pond layout and design. The approach identified residence time as a function of the intermittent nature of stormwater inflows and the flow patterns that develop in the pond during a storm event. Simulations were conducted focusing on two distinct factors; the hydrology (the temporal distribution of the inflows) and the hydraulics (the flow patterns that develop in the pond during a storm). Triangular inflow hydrographs were routed through rectangular basins with equal pond volumes but varying length to width ratios (from 0.5 to 8.0). The model HYDRA3, a two dimensional numerical model was used to solve the depth- averaged flow equations. A second model TRANS was used to simulate the flow through and mixing of the inflow in the pond. In this second stage inflows were tagged by assigning a concentration of zero to the initial pond volume while a concentration of 1.0 was assigned to new inflows. In this way, inflow distribution and its progress through the pond with time could be graphically presented in a concentration plot.

The HYDRA3 and TRANS models are both finite-difference models written by[*Walker*, 1998] according to the author they are similar to the commercial software packages MIKE21 and RMA2. Validation of the TRANS model was achieved using a mass balance computation which ranged from 88%-97%. Results from the models indicated that short-circuiting is associated with wide ponds (i.e. length to width ratios of 0.5), while in long, narrow ponds (length to width ratios of 4.0) plug flow predominates and short-circuiting is reduced. The study concludes that long, narrow ponds have a better ability to treat stormwater inflows since their design allows them to retain more of the inflow than a wider pond. Therefore the hydraulic performance (and thus the residence time) is dependant upon both the pond design and the size of the inflow event (relative to the size of the pond).

[*Chu et al.*, 2005] present a design procedure for stormwater ponds and wetlands to enable the prediction of loading reductions in suspended solids and nutrients. The study used two commercial models QHM and SWMHYMO to simulate storm volume and flow rate in an 8-cell experimental wetland in Alberta, Canada. This model was calibrated with local performance data from the wetland including the mass removal rate of Total Suspended Solids (TSS) and various nutrients as a function of Hydraulic Retention Time (HRT). In this study HRT, defined as the time taken for a unit volume of water to travel through the detention facility, was not modelled assuming plug flow as the dominant form of flow as is assumed in other studies, which calculate retention time as the peak volume divided by the peak outflow rate. However in this study the ratio of the inflow to the volume of water in the pond is calculated to determine residence time. In this way the fraction of outflow that was produced from the storm event can be calculated, as can the fraction of the outflow that originally comprised the permanent pool.

The work on HRT was conducted primarily using SWMHYMO on an existing stormwater wetland with known outflow characteristics. Using simulated inflow and outflow hydrographs for the 1 in 5 year storm, inflow, outflow and pond volume were calculated for each time step. The results showed that not all of the inflow entering the pond left the pond during the storm. For example for the 1 in 5 year storm 50% of the runoff has an HRT of 140 hours and therefore would not leave the pond until the next storm. For the 2 year and 100 year storm the volume of water that remains in the pond until the next storm is 60% and 12% respectively. A polynomial relationship between the existing HRT data and TSS removal efficiency data was used to determine the percentage removal for the 1 in 5 year storm event. Further details on the modelling of retention time and retention time probability functions can be found in [*Holland et al.*, 2004; *Werner and Kadlec*, 2000].

[*Guo and Adams*, 1999] present a model developed to improve the design and evaluation of storm water quality control basins which uses a statistical solution for estimating flow capture efficiency and average detention time in basins with orifice or weir outflow structures. Flow capture efficiency – the extent to which a basin can contain the total inflow volume, is determined with the estimation of the total spill volume which is calculated by considering both the event spill volume and the carryover spill volume. Taking account of the carryover spill volume enables the quantification of consecutive runoff events. The average detention time is estimated, with due consideration for the variable inflow and outflow rates and the random nature of sequences of antecedent periods and runoff events. Analytical determinations of the average detention time are confirmed by continuous simulation modelling.

Other modelling work focuses on the micro-scale processes which determine the fate of particles washed into storm water basins. [*Krishnappan and Marsalek*, 2002] present a model which predicts the transport characteristics of sediment in a stormwater basin. The processes of flocculation and fine sediment settling are considered. The model was developed due to inadequacies in two current techniques for estimating sediment settling in stormwater ponds and since experiments conducted in a rotating circular flume by Krishnappan *et al.*, (2002) confirmed that pond sediments undergo the process of flocculation under different turbulent shear flows. The first modelling technique, the ideal settling tank concept, is criticised for simulating a uniform distribution of flows, a uniform distribution of suspended solids and discrete particle settling (without flocculation) in a tank with uniform (rectangular) geometry. While the second technique, computational fluid dynamics (CFD) is criticised since its ability to simulate the effects of settling and scouring has not been verified by field data and, according to the authors, the simulation of low velocity fields is questionable. To avoid the perceived inadequacies of these two techniques, [*Krishnappan*, 1990] use an extension of an earlier settling and flocculation model for still water. This model is extended to represent the range of flows that were observed in the rotating flume under laboratory conditions. In the model, sediment motion is considered in two stages; the settling stage is simulated using a 1-dimesnsional unsteady advection-diffusion equation, which determines the balance between the settling flux and the diffusive flux in the vertical direction. The flocculation stage is simulated using a coagulation equation describing the number-concentration balance of particles that aggregate due to collision. This latter stage takes into account electrochemical properties of the sediment/water mix and chemical/organic coatings of the particles etc. The model predictions agreed well with measured suspended sediment concentrations-time curves from experimental data. Furthermore comparisons of the predicted size distribution of the flocculating sediment with the measured data also showed good agreement.

One further study worth mentioning here is that of [*Ellis et al.*, 1995]. They develop a model which considers particle-size, settling velocities and hydraulic retention time for estimation of pollutant removal efficiency in flood storage reservoirs The model considers sediment removal efficiency under varying hydraulic conditions, basin geometry and inflows. The paper outlines a simple modelling basis for estimating the pollutant removal efficiency for flood storage reservoirs receiving drainage water from impervious urban surfaces. They produced estimates of sediment capture or trap efficiency for a 2 year design storm. The design of the sediment model described in Chapter 5 of this thesis uses a similar approach to that of [*Ellis et al.*, 1995].

3.2 Introduction to Stormwater Management Basin Modelling

The model applied and tested within this thesis was primarily designed as a tool to enable the simulation of flow in retention ponds and detention basins and was later developed to include the simulation of sediment removal for water quality enhancement [*Wallis et al.*, 2006]. The model was designed and coded by Dr. Steve Wallis and subsequently tested and applied to simulate pond performance as part of this thesis. The details of the flow model are described and validated in the following sections.

3.2.1 The Flow Model

The flow model was written using Microsoft Excel. The model simulates several pond geometries with various possible outlet configurations that include the choice of a high-level v-notch weir and/or a low level submerged pipe. The configuration of the pond is determined by the radius, the initial water level in the pond and the number, elevation and configuration of the outlets (e.g. the weir angle and the pipe diameter). Although the pond geometry simulated in this work is always cylindrical, other pond shapes are considered elsewhere [*Wallis et al.*, 2004].

Flow in the pond is determined by equation 3.1 where the rate of change of volume in the pond ($\frac{dv}{dt}$) is equal to the flow entering the pond (Q_i) minus the flow leaving the pond (Q_o). This describes a mass balance for the water.

$$\frac{dv}{dt} = Q_i - Q_o \qquad (3.1)$$

The inflow to the pond Q_i is given using either a triangular inflow hydrograph or could be derived from a real rainfall dataset. The outflow from the pond Q_o is calculated as shown below, depending on whether outflow occurs through a weir (equation 3.2), a submerged pipe (equation 3.4) or both.

$$Q_O = C_W H_W{}^{5/2} \qquad (3.2)$$

Equation 3.2 describes flow through a v-notch weir where Q_o is the flow, H_w is the head over the weir and C_w is the weir coefficient given by:

$$C_W = \frac{8}{15}\sqrt{2g}C_{dW}\tan\left(\frac{\phi}{2}\right) \tag{3.3}$$

where g is the acceleration due to gravity, C_{dw} is the weir's coefficient of discharge and ϕ is the angle of the weir [*Chadwick and Morfett*, 1998].

$$Q_O = C_P H_p^{1/2} \tag{3.4}$$

Equation 3.4 describes flow through a submerged pipe, modelled as a small, submerged orifice, where H_p is the head over the pipe and C_p is the pipe coefficient given by:

$$C_P = \sqrt{2g}C_{dp}A_p \tag{3.5}$$

where g is the acceleration due to gravity, C_{dp} is the pipe's coefficient of discharge and A_p is the cross-sectional area of the pipe [*Chadwick and Morfett*, 1998].

Equation 3.1 is solved using a standard storage routing method [*Shaw*, 1997] in finite difference form, and employing Newton-Raphson iteration to deal with non-linearities. Further details appear in [*Wallis et al.*, 2006].

3.3 Flow Model Verification

3.3.1 Comparison with Commercial Software

The cylindrical pond model was compared with a model developed using ERWin, a modelling tool used specifically for the simulation of flow in SUDS. To ensure that comparisons were appropriate, a few adjustments were made to the existing models. Firstly, a storm hydrograph was constructed in ERWin and imported into the pond model, to ensure both models were receiving the same volume and timing of inflow. Secondly, the outflow structure, in this case a 1 metre-wide rectangular weir, was made exactly the same in both models. Finally, the diameter of the outlet pipe in the ERWin model was set to zero, to ensure that all of the outflow passed over the weir.

Results

The comparison of the cylindrical pond model with the ERWin model provided very encouraging results. The output from both models compared well, producing almost identical results for the same inflow hydrograph (Figure 3.1)

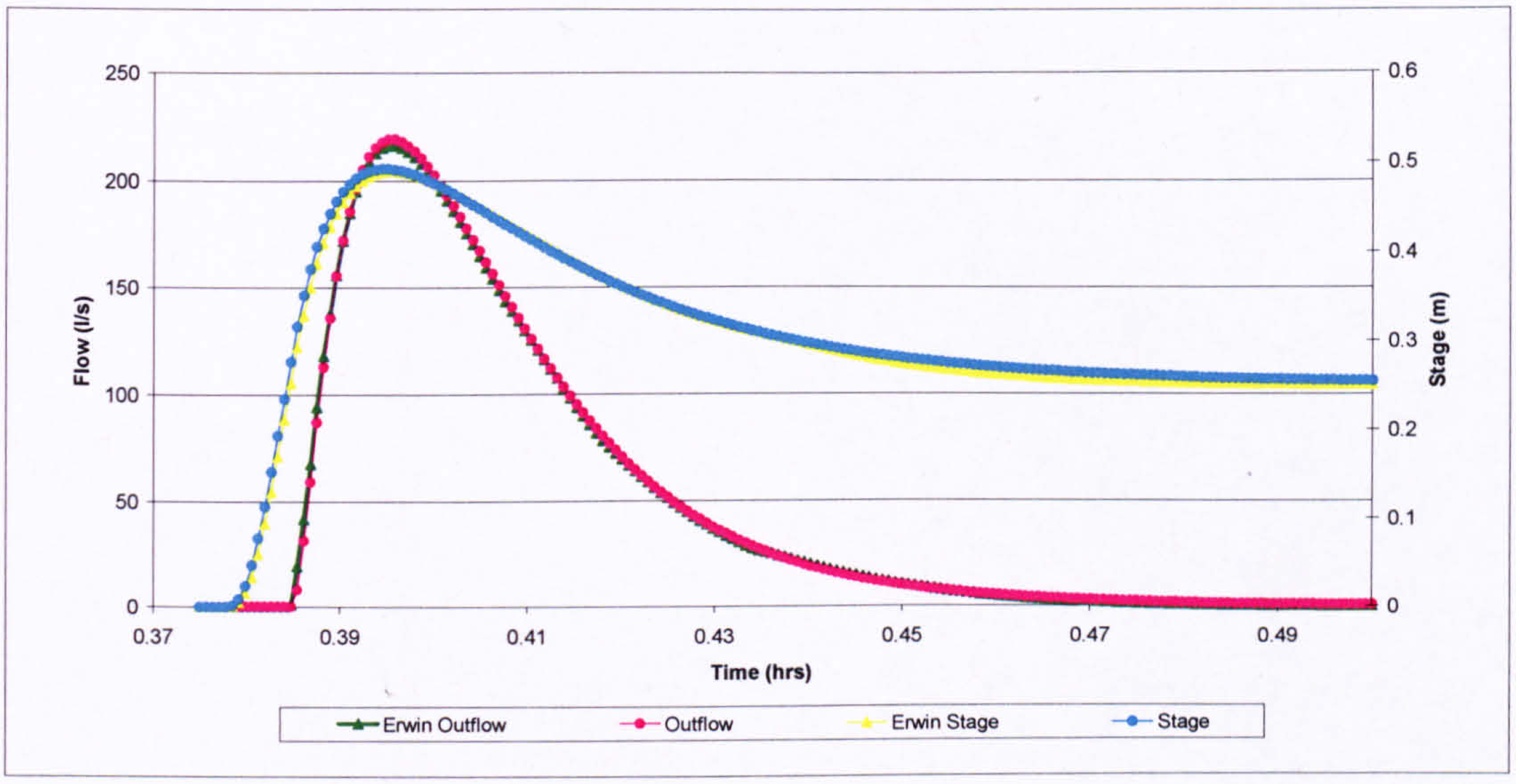


Figure 3.1: Comparison of cylindrical pond model and ERWin, showing comparison of simulated outflow and stage.

3.3.2 Comparison with Observations from an Existing Retention Pond

The cylindrical pond model was further verified using monitored inflow and outflow data from Claylands Pond, a functioning SUDS basin near the village of Ratho to the west Edinburgh. It was not originally designed as a retention pond, but rather built as an irrigation pond by a local farmer in 1973 [*McClean*, 1998]. Due to this Claylands pond is in a natural state, being formed from earth banks on all four sides and having no artificial lining. The pond was first used as a SUDS pond with the construction of the M8 motorway which runs adjacent to Claylands pond, since a condition of the construction required drainage from the road to pass through the pond for treatment.

Figure 3.2 shows a plan of Claylands pond. As can be seen from Figure 3.2, the pond is rectangular in shape with three different sources of inflow entering through a thin plate weir. First, runoff from local sources is discharged into the pond, an un-named tributary also discharges runoff into the pond and finally stormwater runoff from the motorway is discharged to the pond via filter drains that run the length of the road. Water leaves the pond via a v-notch weir at the outlet. The volume of Claylands pond was estimated to be $2070m^3$ after a depth analysis was undertaken by [*McClean*, 1998].

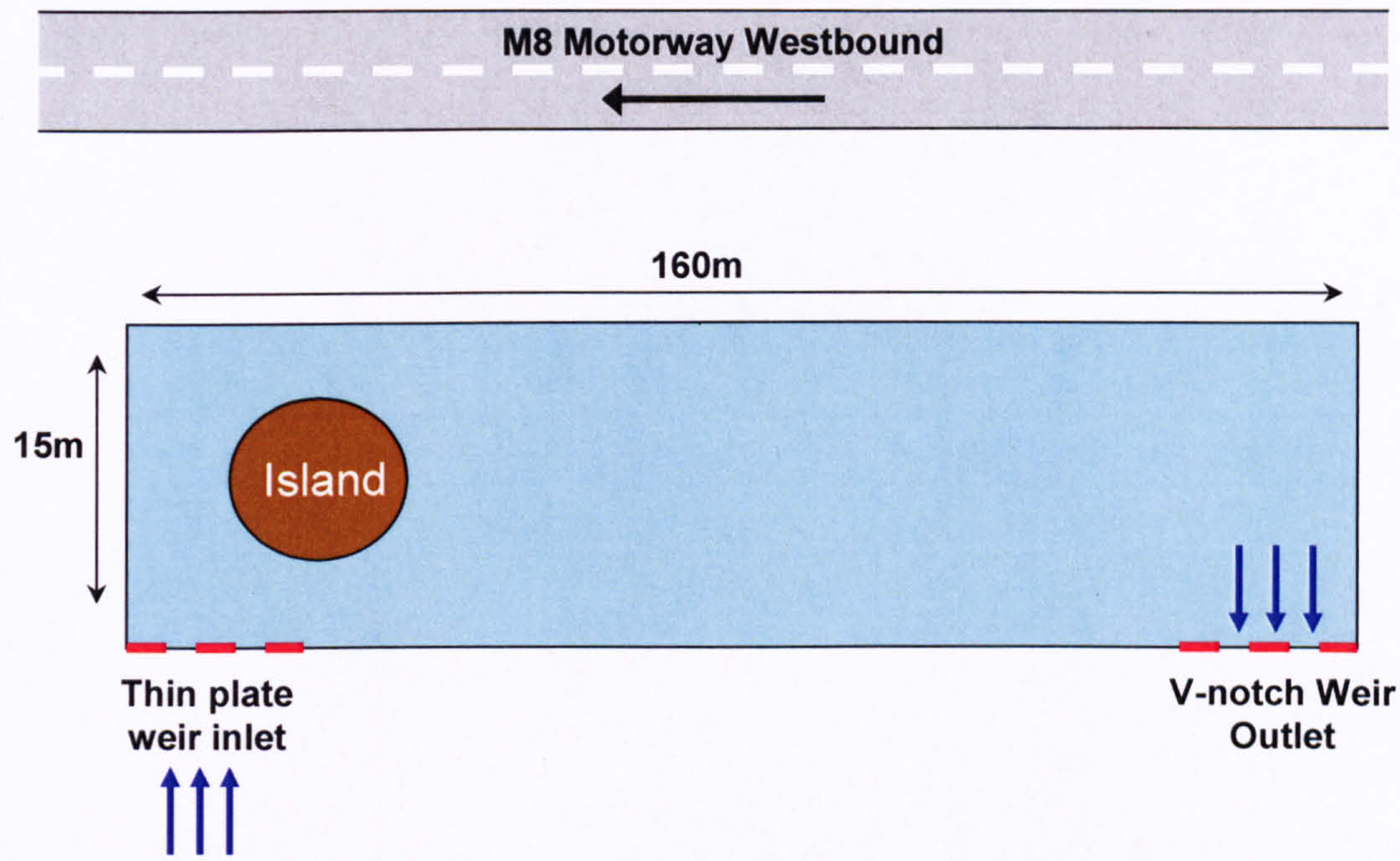


Figure 3.2: Plan of Claylands pond showing inlet, outlet, island and nearby M8 motorway (adapted from McClean 1998)

3.3.3 Modelling Claylands Pond

Claylands pond was modelled as an equivalent cylinder of radius 27.6m. Flow data for 4 storm events were obtained from SEPA and imported from Hydrolog into excel [*McClean*, 1998]. The inflow data for each of the four storm events were run through the cylindrical pond model and graphs showing modelled outflow and observed outflow were produced (Figure 3.3).

Results

Figure 3.3 and 3.4 shows the observed outflow data from Claylands pond alongside the model predictions for two of the storms. The storms are extremely well predicted in shape and volume. However the simulations show that the model does not predict a sufficient time delay i.e. the simulated outflow is too early. There is a clear time lag of approximately

2 hours between the predicted and observed outflows. The absence of a time lag in the model occurs because the model is based on level pool routing in which the whole surface of the pond rises or falls simultaneously. However Claylands pond, which has a large length to width ratio, behaves more like a river in which a change in surface elevation can take some time to propagate along the system. Other ponds considered later in this thesis, do not have large length to width ratios, hence, the inability to simulate such time delays is not an issue. Furthermore, in assessment of real pond performance, it is primarily the magnitude of the peak flow that is considered in UK design targets rather than the timing of the outflow peak.

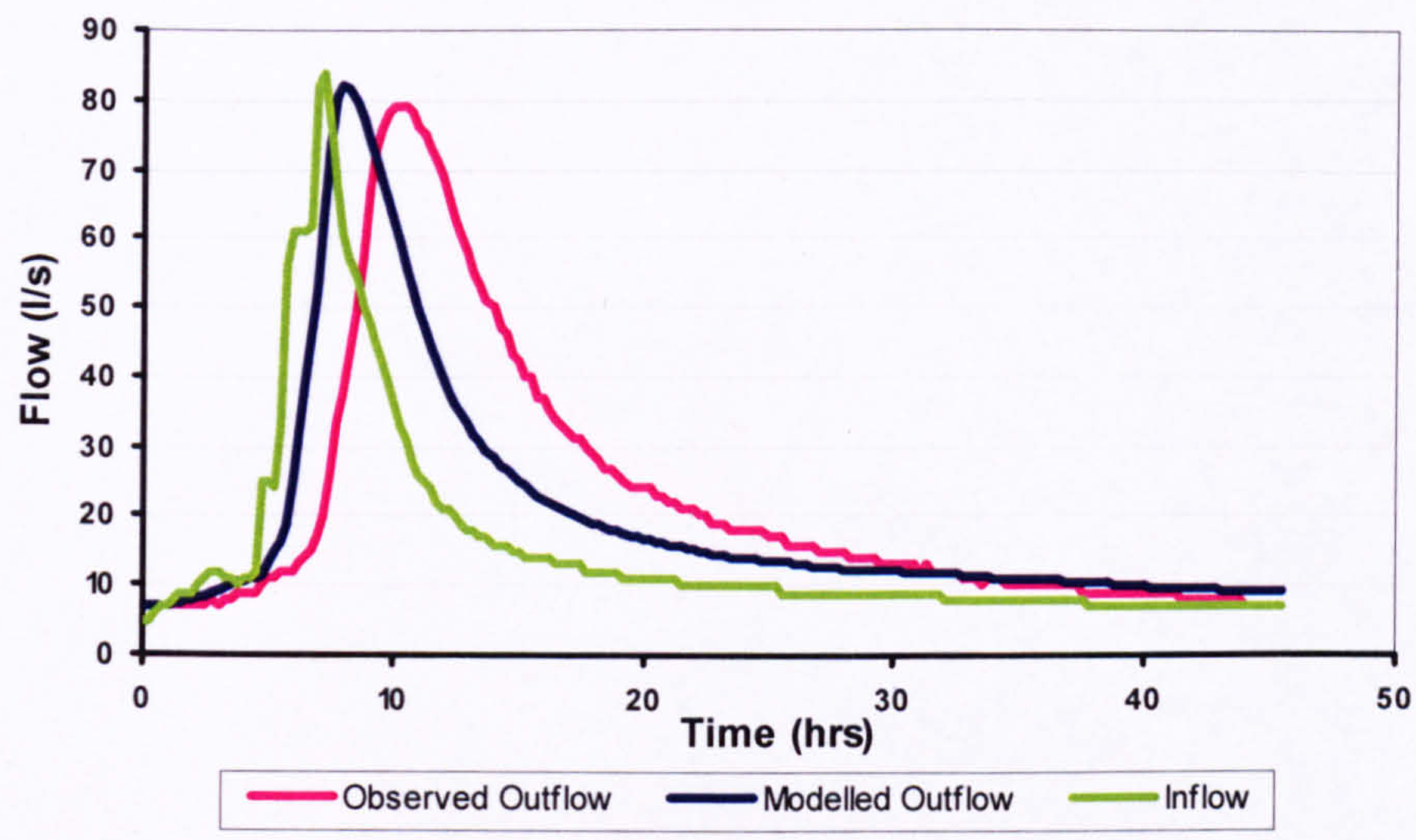


Figure 3.3: Showing observed Inflow, observed outflow, and modelled outflow for a storm at Claylands pond

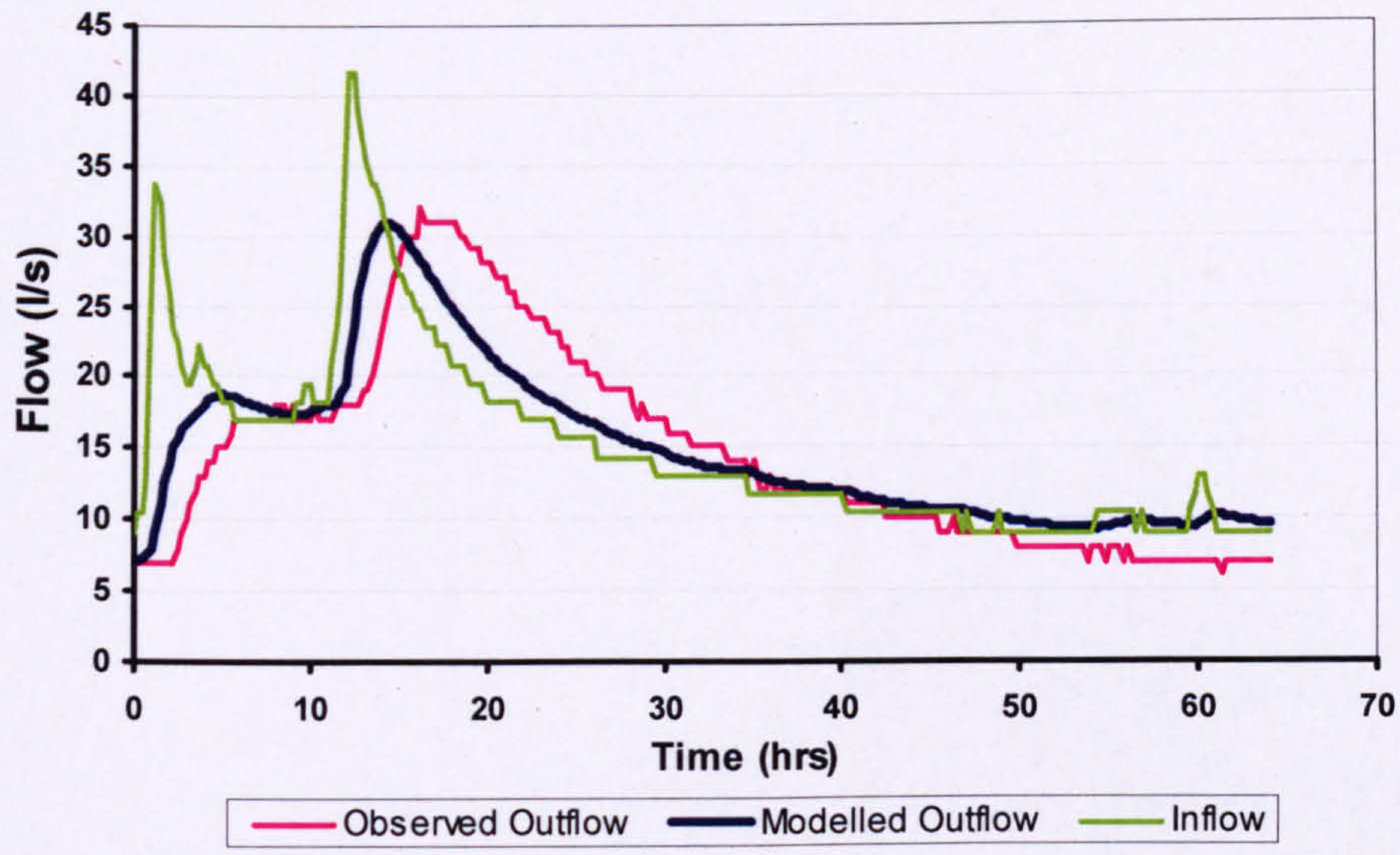


Figure 3.4: Showing observed Inflow, observed outflow and modelled outflow for a storm at Claylands pond

3.4 Initial Pond Model Simulations

3.4.1 Sensitivity Analysis of the Cylindrical Pond Model

Having completed an initial verification of the pond model in section 3.3, a sensitivity analysis was performed using the cylindrical pond to gain a better understanding of pond dynamics, the major influences affecting flow attenuation behaviour in retention ponds and to ensure that the simulations remained physically realistic.

Inflows to the pond were constructed using an isosceles triangular inflow hydrograph with a peak flow of 50 l/s, a storm duration of 3.2 hours and a total inflow volume of 288 l. Sensitivity was investigated by observing the changes in several flow attenuation performance indicators when values of four pond parameters were varied. The parameters that were varied in the study were pond Radius, Weir Crest Elevation, Weir Angle and the Initial Water Level in the pond before the onset of a storm event (Figure 3.5). Varying the radius changes the surface area of the pond, whilst varying the weir crest elevation and the initial water level alters the storage capacity of the pond above the permanent pool level. Throughout the rest of the thesis this storage capacity available above the permanent pool is referred to as the Temporary Storage Volume (TSV). Changing the weir angle affects the flow capacity of the weir. In the simulations, each of these parameters was varied independently to determine its effect on pond performance. Indicators of performance used were Peak Flow Ratio and Peak Time Delay. The former is the ratio of peak outflow to peak inflow, the latter is the time lag (in hours) between Peak Outflow and Peak Inflow. A small value of the peak flow ratio indicates a good attenuation performance. For example a peak flow ratio of 0.1 indicates that the peak outflow is 10% of the peak inflow. Whereas a peak flow ratio of 0.6 indicates a much larger peak outflow, and hence a poorer performance.

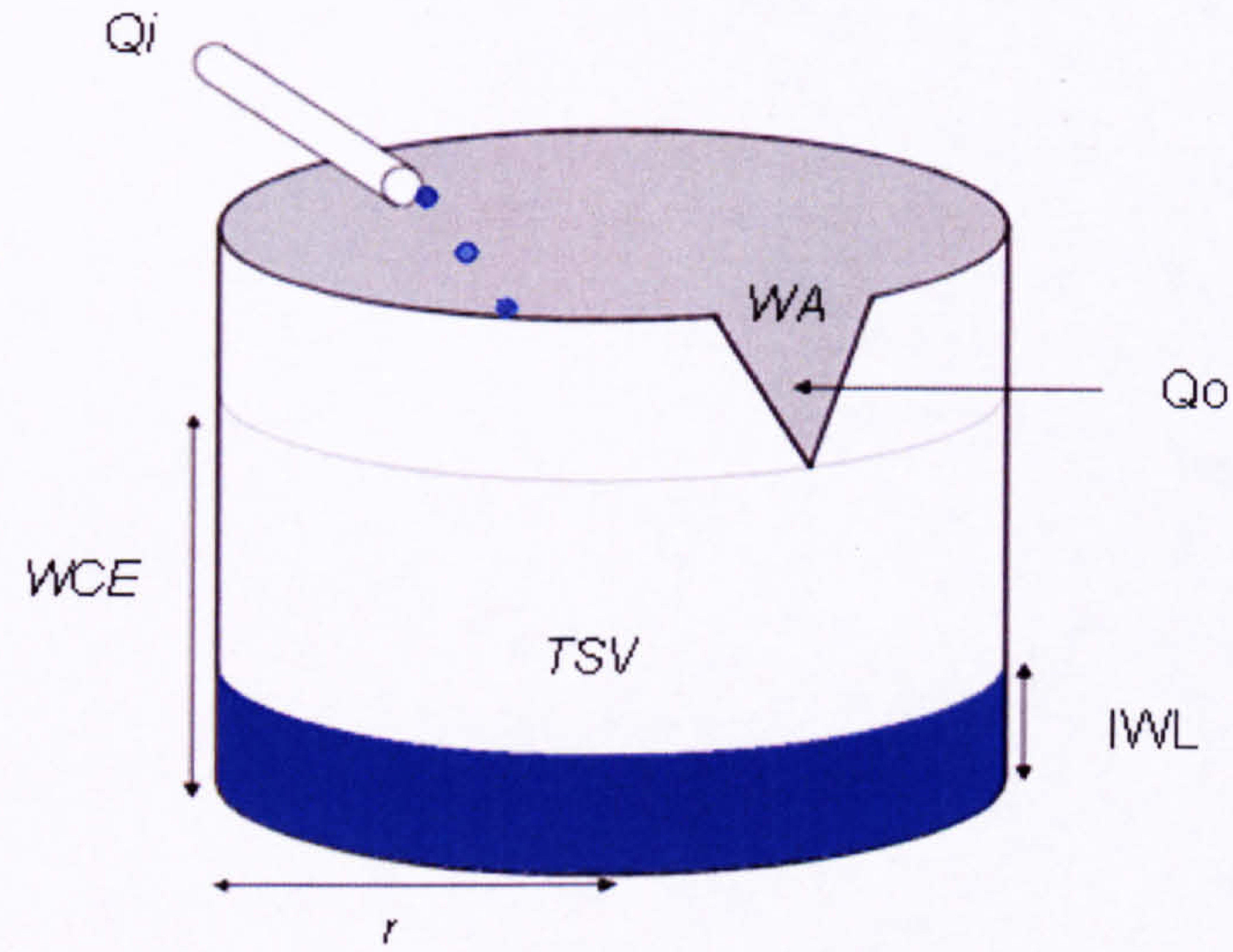


Figure 3.5: Diagram of a cylindrical pond showing inflow (Qi), Outflow (Qo), Radius (r), Weir Angle (WA), Weir Crest Elevation (WCE), Initial Water Level (IWL), and Temporary Storage Volume (TSV). The permanent pool level is determined by the IWL and the elevation of the weir crest.

The following sensitivity analysis was undertaken on a standard pond with the following baseline properties: Radius of 10m, Weir Angle of 90°, Weir Crest Elevation of 2m and an Initial Water Level of 2m. A time step of 1.5 minutes was used in all simulations, based on the results of a time step analysis undertaken to ensure that accurate solutions were being obtained, i.e. convergence was achieved in the sense that further reduction of the time step yielded no further change in the results. The results are discussed in the following subsections.

3.4.2 Weir Crest Elevation

In this analysis four Weir Crest Elevations were tested, ranging from 0.5m to 2m. In all cases, the IWL remained fixed at 0.5m. The results are given in Figure 3.6 and the Peak flow ratio decreases with increasing Weir Crest Elevation and Peak Time Delay increases, indicating an improvement in flow attenuation performance of the pond. As the IWL is fixed at 0.5m, raising the weir further up the pond increases the available storage (TSV) for incoming stormwater. As a consequence total outflow from the pond decreases, because more of the inflow can be stored below the weir crest.

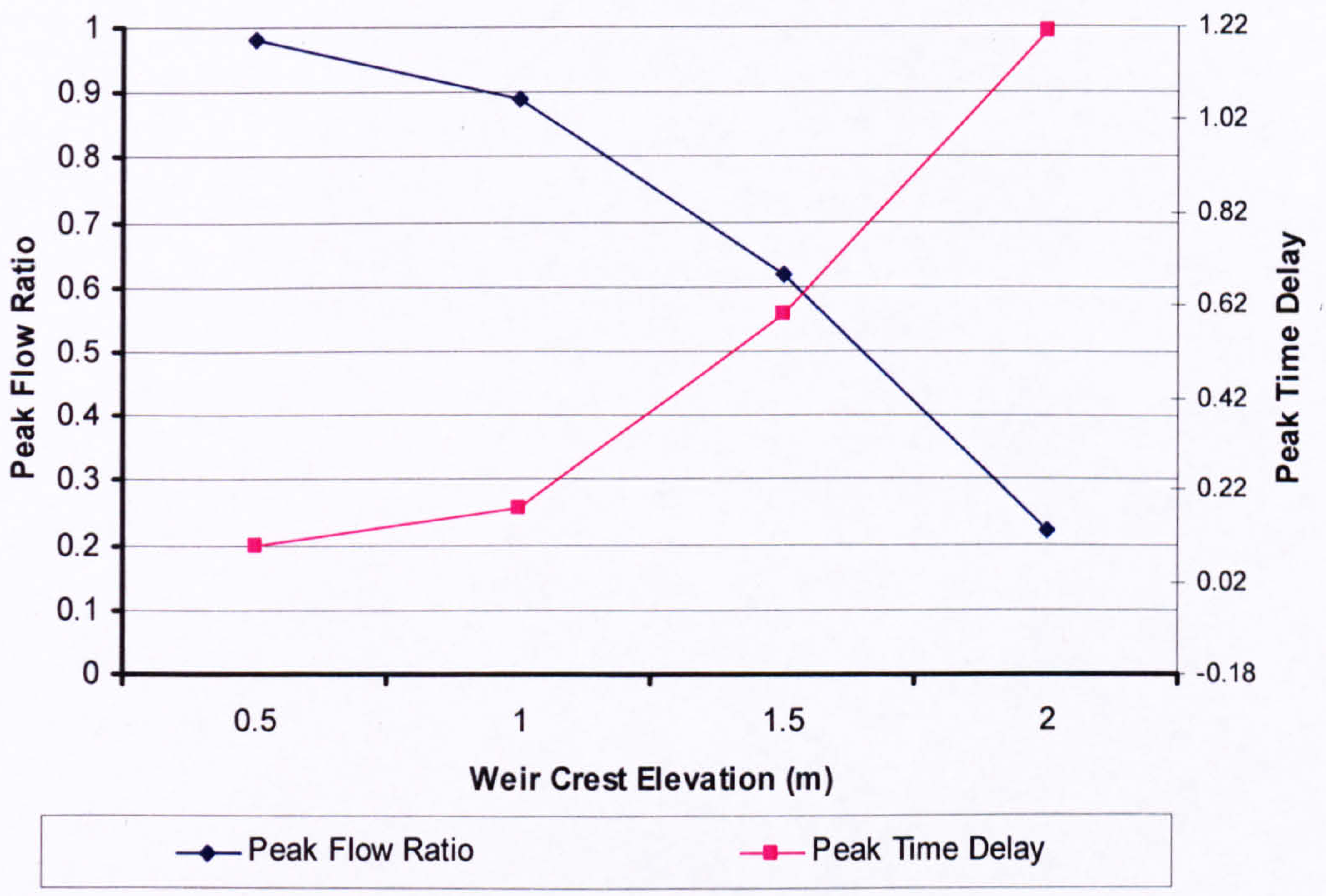


Figure 3.6: Peak flow ratio and Peak Time Delay with increasing Weir Crest Elevation

3.4.3 Initial Water Level

In this analysis four Initial Water Levels were tested ranging from 0.5m to a maximum of 2m (since maximum initial water level is dictated by the elevation of the weir crest). Figure 3.7 shows that as Initial Water Level increases further up the pond, the Peak flow ratio increases and Peak Time Delay decreases. This is an undesirable change, indicating a reduction in flow attenuation which reflects the reduction in TSV that occurs as higher initial water levels displace temporary storage capacity in the pond. The decline in attenuation is also indicated by an increase in total outflow volume as initial water level increases. This is simply related to the decreasing TSV available at the start of the storm as the IWL is increased.

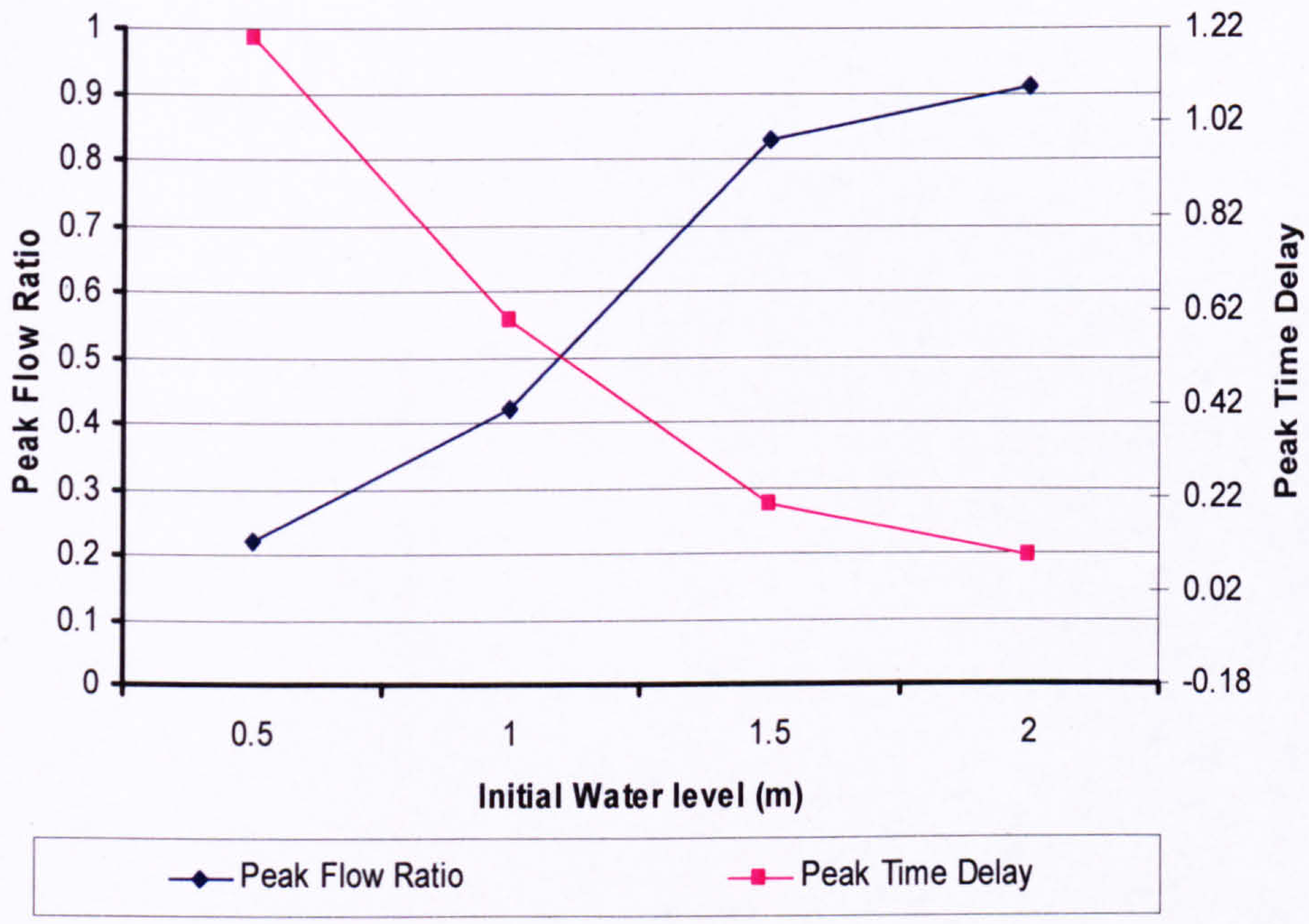


Figure 3.7: Peak flow ratio and Peak Time Delay with increasing Initial Water Level

3.4.4 Sensitivity to Radius

Pond radius was varied in the range 5 - 25m. The results in Figure 3.8 show that as radius increases Peak Flow Ratio decreases, while Peak Time Delay increases, which corresponds to an improvement in pond performance with increasing radius. Unlike the previous two cases, this improvement does not occur due to any change in TSV (since TSV is zero for all five radii). Instead, the improvement comes from reduced water levels that occur because the fixed inflow volume is distributed over an increasing surface area, as the radius is increased. The water level provides the head that drains the pond through the weir. Clearly, reduced heads create smaller outflows, so smaller peak outflows are associated with larger pond radii.

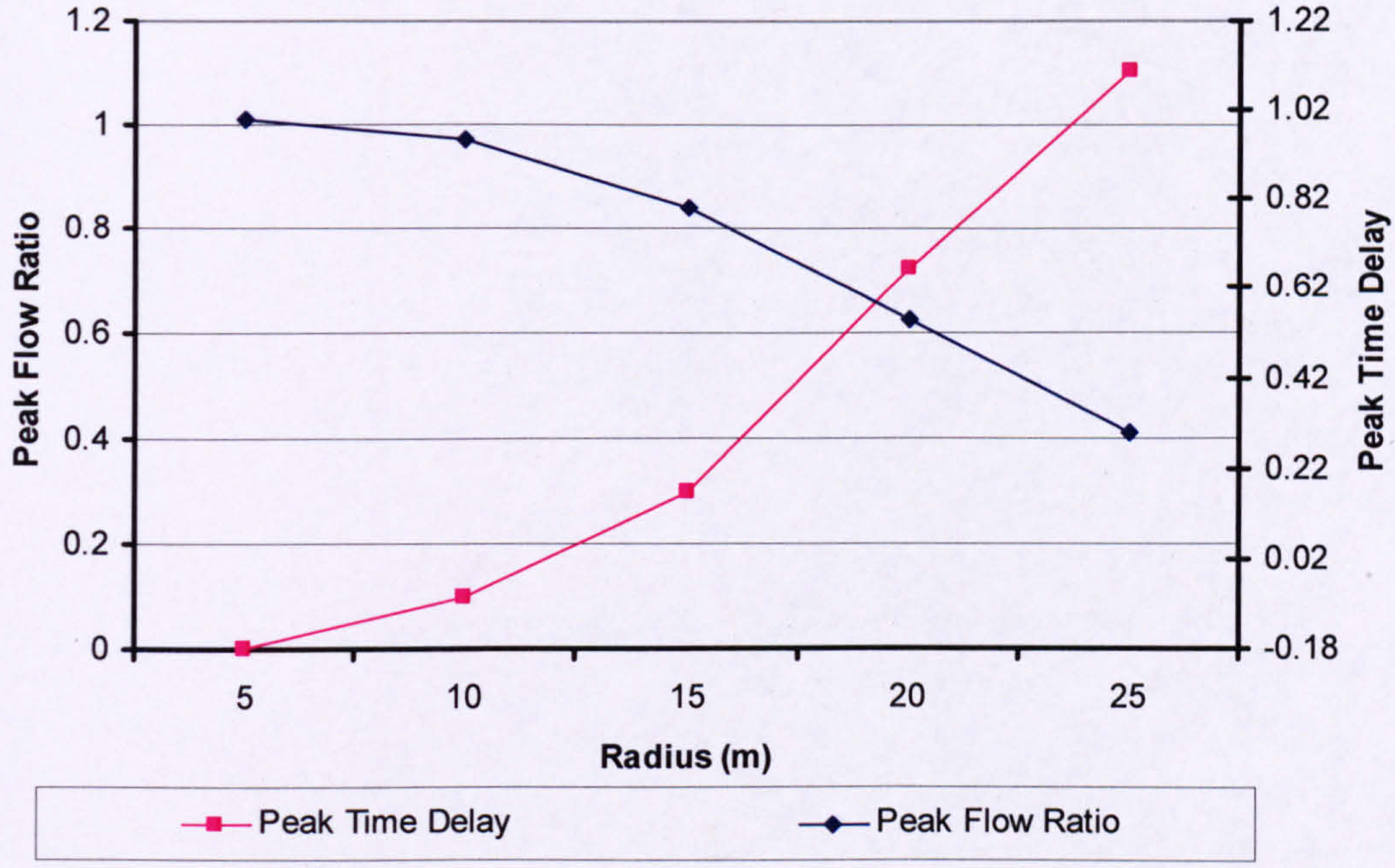


Figure 3.8: Peak flow ratio and Peak Time Delay with increasing Radius

3.4.5 Weir Angle

Five values of weir angle were tested ranging from 45°-120°. The results in Figure 3.9 show that Peak Flow Ratio increases with increasing weir angle whilst Peak Time Delay decreases, indicating a decline in performance. Again, the change in performance is concerned with the operation of the weir, rather than TSV (since TSV is zero in all five weir angle cases). With increasing weir angles, greater outflow occurs through the weir leading to smaller storage of water above the weir crest and lower water levels. Note that the change in performance caused by variation of the weir angle is significantly smaller than the change caused by varying the other three parameters.

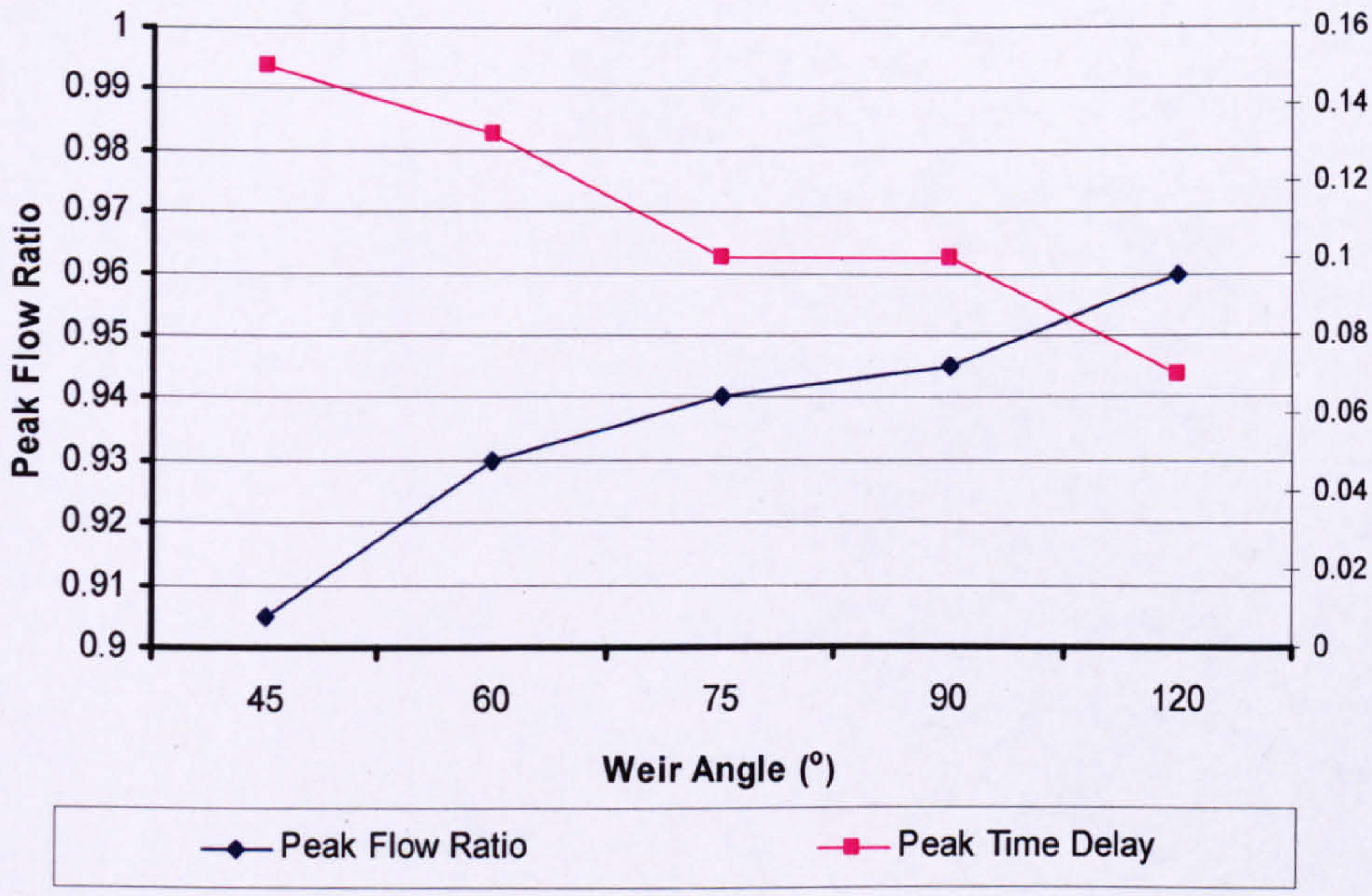


Figure 3.9: Peak flow ratio and Peak Time Delay with increasing Weir Angle.

3.4.6 Discussion

In all cases greater TSV allows greater storage of inflow below the weir crest, reduces total outflow volume and improves flow attenuation (reducing peak outflows and delaying the time of the outflow peak). Although similar improvements in flow attenuation may also be achieved by increasing the pond radius, this requires the use of a lot of land, which is not attractive to developers. Modifications to the weir angle have comparatively little effect on flow attenuation.

The criticality of the temporary storage volume has long-term implications for retention ponds. Over long periods sediments will gradually accumulate on the bed of the pond, reducing temporary storage and will eventually compromise a pond's ability to attenuate flows. This is illustrated by the deterioration in flow attenuation performance that occurs when TSV is reduced. A further implication of this is that short dry periods between storms will adversely affect attenuation performance, since large TSV will only be provided if the pond has time to empty after each storm event. Due to the configuration of the outlet (a single, high set weir) the pond can only drain down to the crest of the weir after each storm, leaving no TSV for incoming stormwater in subsequent storm events.

Undoubtedly this will adversely affect pond flow attenuation performance. Later simulations investigate the effect of incorporating a secondary outlet, a submerged pipe in the pond outlet configuration.

3.4.7 Conclusions

Investigations in this chapter have shown that the flow model is able to simulate flow in retention ponds. A sensitivity analysis on a generic pond indicated that the TSV is a critical parameter in determining pond flow attenuation.

In this chapter, only single storm events are considered. However, in order to accurately asses the performance of an operational pond, it is necessary to simulate hydrological conditions where the pond is subject to a sequence of storms and to investigate how the pond performs in reducing peak flows and removing sediment under these conditions. These issues are examined in Chapters 4, 5 and 6 of the thesis.

4 Model Application to Investigate Retention Pond Design for Improved Flow Attenuation Performance

Using the pond model developed in Chapter 3, this chapter assesses the influence of storm magnitude and antecedent period on the flow attenuation performance of retention ponds, and demonstrates the importance of designing for multi-event storm sequences. The findings are applied to a retention pond in current use in East Scotland, taking into consideration likely changes in climate, these simulations result in proposed design improvements to enhance current and future performance of the pond.

4.1 Introduction: Retention pond design

Retention ponds are stormwater storage basins, which have a permanent pool of water all year round. As discussed in Chapter 2, they attenuate flows by providing an area of temporary storage during storm events, which helps reduce peak outflows and delays the outflow hydrograph (as demonstrated in Chapter 3). Hence, they can be used to counter the negative effects of urban developments, which may increase peak runoff flows to 2-5 times the pre-development magnitude [*Crippen*, 1965; *Espey et al.*, 1969].

Criteria for retention pond design vary throughout the world, however, the designer's broad aim is that retention ponds should constrain post-development flows to the pre-development levels, i.e. runoff hydrographs should be similar before and after development, ensuring that peak flows and their timing are not adversely affected by urbanisation. In some countries more specific criteria are used. In this regard, Tables 2.1 presented in Chapter 2 summarises a selection of design standards from across the world that have been developed to comply with national and international guidelines. A theme that emerges from these is that a pond should attenuate flows for a single design event storm, however, even amongst the small selection in Table 2.1 there is wide variation in standards. Furthermore, there appears to be no clear definition of the level of attenuation that should be provided by ponds.

Retention ponds are often required to be effective in storing storm events of a various frequencies (since the design storm for an area undergoing urbanisation can range from the 1 in 10 to the 1 in 100 year storm), and must be sized properly at the design stage to protect against catchment flooding. However because they have to fulfil a dual function, they must also be able to treat smaller rainfall events - which are much more important in terms of water quality deterioration [*Hingray*, 2002; *Pitt*, 2005].

As discussed in Chapter 2, this requirement to attenuate a range of storm sizes makes pond design a complex procedure. According to the design guidance available in the UK and the USA (Table 2.3, Chapter 2) current practice encourages pond design based on the attenuation of a single storm event of a specified magnitude taking no account of prior storm events. However, in reality a pond's performance will be determined by its ability to attenuate and drain sequences of storms of varying magnitude and frequency.

Pond geometry and the location and design of inlets and outlets are also important in terms of flow attenuation. As shown in Table 2.3 (Chapter 2), countries such as Canada and the US promote the use of multiple outlets to enable ponds to manage a range of storm magnitudes. This is in contrast to the type of pond design in current use in the UK, where the use of a single-level outlet device is advised (Figure 4.1).

The simulations in Chapter 3 highlighted inherent weaknesses in flow attenuation in retention ponds that only have outlet device(s) at the design elevation of the permanent pool. Unless there is a long time between storm events allowing evaporation or infiltration to lower the water level of the permanent pool, no temporary storage volume is available. During most storm events these ponds only store water in the volume provided above the outlet, and therefore provide little flow attenuation. Surprisingly, in the UK there are a number of retention ponds that operate like this, perhaps, in part because the primary design guidance, CIRIA's Design Manual for Scotland and Northern Ireland (2000), does not specify the use of multiple outlets at varying elevations (Figure 4.1).

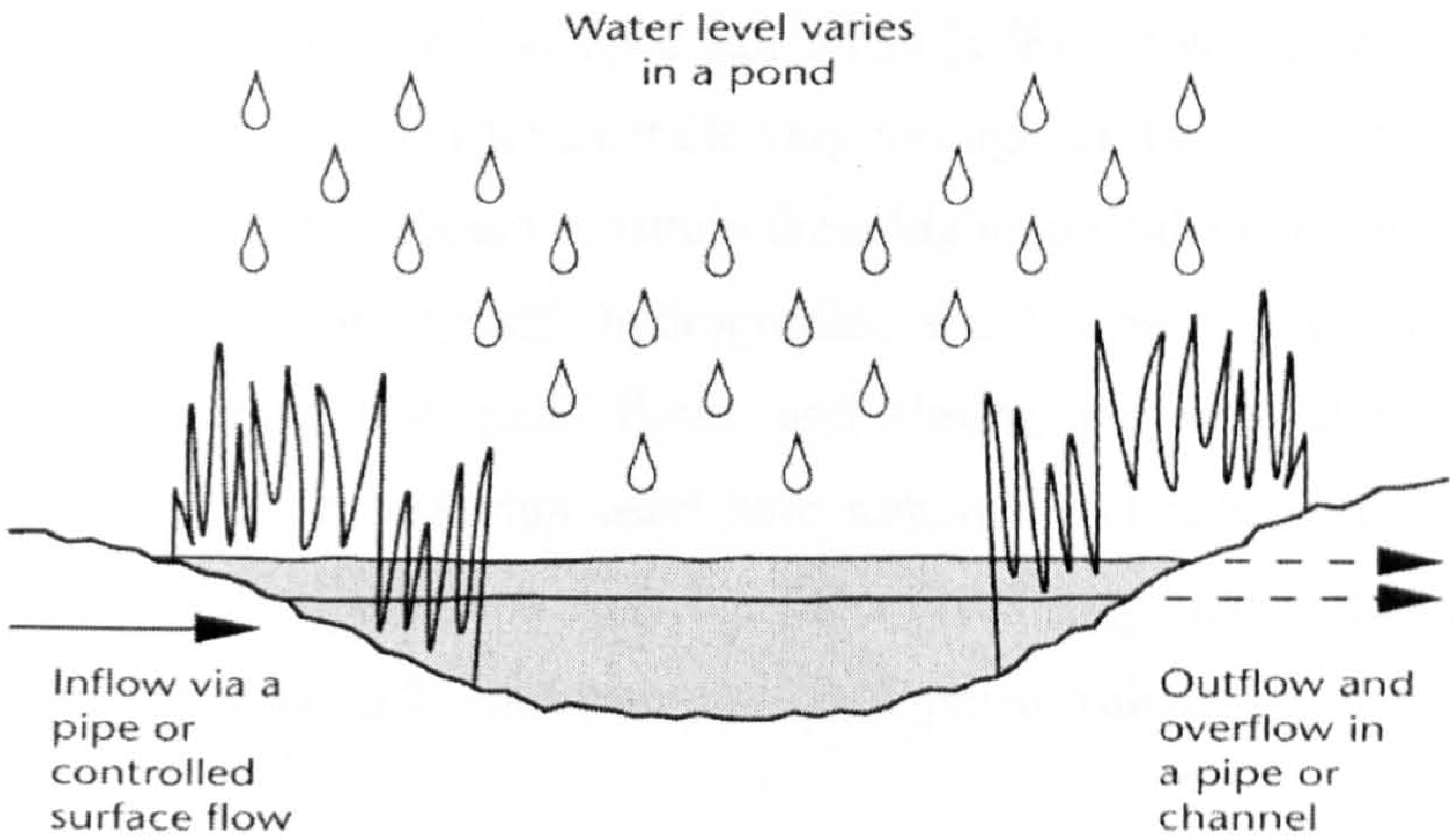


Figure 4.1 Typical retention pond design (CIRIA, 2001*)

A final design consideration is that retention ponds should provide some water quality enhancement. This occurs through the permanent body of water providing large residence times, which encourages the settling out of pollutants. Also, where ecological habitats evolve, biological activity may further improve water quality [*Butler and Davies*, 2000; *CIRIA*, 2000]. It is far from clear, however, to what extent designing for water quality enhancement (slow drainage, promoting long residence times) may conflict with designing for flow attenuation (rapid drainage, providing storage for the next storm). This is investigated in Chapter 6, whilst the current chapter focuses on flow attenuation alone.

4.2 Appropriate design scenarios for retention ponds

As discussed earlier, there appears to be no widely accepted definition of the degree of flow attenuation that a retention pond should provide. For the purpose of this study, therefore, a pond failure criterion was defined that is based on a specified impact of urbanisation. Since many ponds are designed to reduce peak discharges to those of the pre-developed site, 'good flow attenuation' or attenuation that reduces peak discharges to a satisfactory standard is defined as that which reduces peak outflow to 50% of the peak inflow. Research has also shown that urbanisation adversely affects the timing of peak runoff; however since much of the design guidance encourages pond design based only on peak flow reduction, it is only the magnitude of the peak flow that is considered here. Consequently, in the remainder of this chapter, a pond is considered to have 'failed' if the peak outflow is greater than 50% of the peak inflow. This threshold was chosen since early research on catchment hydrology concluded that urbanisation could increase peak flows by

2–5 times those from the pre-developed catchment [*CIRIA*, 2000; *Crippen*, 1965; *Espey et al.*, 1969]. Although design criteria details vary throughout the world, the designer's broad aim is that retention ponds should constrain these higher post-development flows to the pre-development levels, i.e. runoff hydrographs should be similar before and after development, ensuring that peak flows and timing are not adversely affected by urbanisation. The failure criterion used here assumes that urbanisation has caused the minimal impact to the hydrograph from the alternatives suggested in the literature i.e., that peak flows are increased to 2 times their pre-development value.

The remainder of this chapter presents a modelling investigation in which a hypothetical pond was designed to provide a 50% reduction of the peak flow for the 1 in 25 year event. This design scenario was intended to be representative of typical pond designs currently used in the UK (Table 2.3, Chapter 2). This is followed by an investigation of pond performance under smaller (1 in 2 year) consecutive events with varying antecedent periods. This approach enabled the influence of initial pond level on flow attenuation to be assessed and allowed improved design criteria for retention ponds to be proposed.

4.2.1 Retention pond design for single and multiple events

Based on the 50% failure criterion defined earlier, simulations were conducted to determine the size of a cylindrical pond that would achieve 'good flow attenuation', initially using a single 90° v-notch weir outlet at an elevation 5m above the base of the pond. The pond was designed to be the minimum size required for the attenuation of a 1 in 25 year storm. This event had a peak inflow of 250 l/s and a duration of 24 hours. In the UK such an inflow might be typical of a 1 in 25 year event for a pond draining an urban area of 1-2km^2*. However, any size of storm could be selected and would simply result in a different pond radius. Simulations showed that a pond radius of 75m was required to achieve a 50% peak flow reduction of the 1 in 25 year event, assuming a single outflow weir with a 90 V-notch weir and that the pond was fully drained to the weir crest prior to the event. This is a typical design based on the guidelines from [*CIRIA*, 2001]. Inflow and outflow hydrographs for this are shown in Figure 4.2.

* See Appendix C

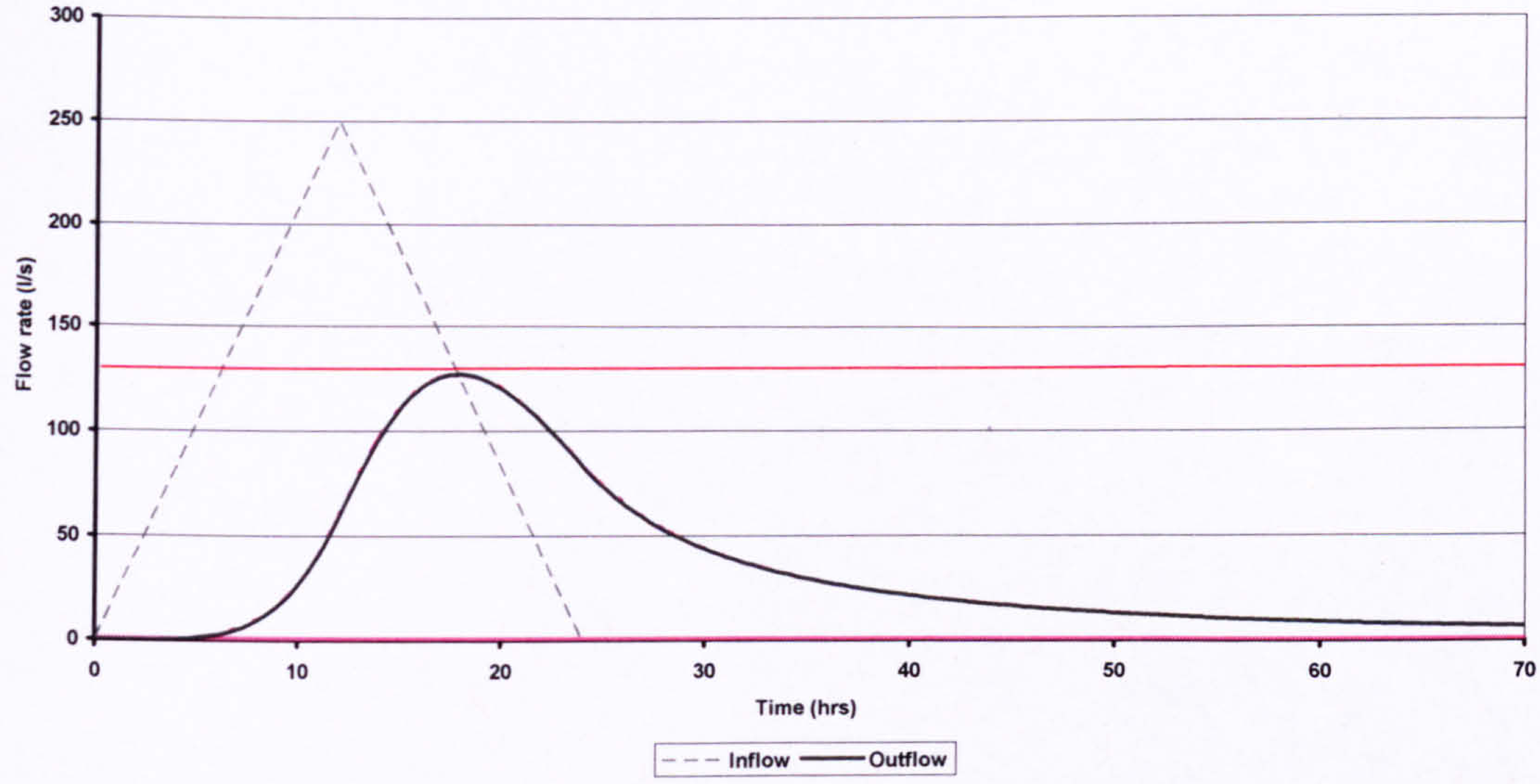


Figure 4.2: Flow hydrograph showing a 50% reduction in peak outflow for the 1 in 25 year storm event in a 75m radius cylindrical pond

In order to test the above pond's performance under one or more consecutive 1 in 2 year events, it is first necessary to decide on the relative magnitude of a 1 in 2 year event. Figure 4.3 shows combined flood frequency curves for Scotland and the U.K [*Fleming*, 2001]. Flood frequency curves are used to depict the number of times per year on average, that floods of a given magnitude are equalled or exceeded. The data presented in Figure 4.3 is derived from two sources; a regional flood frequency analysis for Scotland [*Biswas and Flemming*, 1966], the data points for which are represented by "plus" symbols, and a regional flood frequency analysis for the UK, [*NERC*, 1975], represented by the "dots" and "crosses". As data are collected from different river basins, the y-axis is normalised by mean flow. To interpret the x-axis, a secondary axis has been provided that shows the return period, T, in years. Although regional flood frequency curves for Scotland were produced by [*Biswas and Flemming*, 1966] independently of (and earlier than) those produced for the whole of the UK in the Flood Studies report [*NERC*, 1975], according to [*Fleming*, 2001] both curves compare well.

The flood frequency curve in Figure 4.3 was used to estimate the approximate scale of a 1 in 2 year inflow event in relation to a 1 in 25 year event in the UK. The figure shows that the inflow of the 1 in 2 year inflow event is approximately half the magnitude of the inflow of the 1 in 25 year inflow event [*Fleming*, 2001]. Consequently, it was assumed that for the pond design illustrated in Figure 4.2 (showing the 1 in 25 year inflow event of 250 l/s), the corresponding 1 in 2 year event peak inflow is 125 l/s, i.e. half the magnitude of the 1 in 25 year inflow event.

A simulation using the 75m radius pond (configured above to produce a 50% reduction in peak flow for the 1 in 25 year design storm), achieved a 69% reduction for a single 1 in 2 year storm, which is an example of good attenuation, since it exceeded the 50% reduction in peak by a further 19%. However, as discussed earlier, a pond's flow attenuation performance does not only depend on the attenuation of a single storm, but also on its ability to attenuate a sequence of storms. Consequently, multiple rainfall events and the effect of the duration of the antecedent period between storms were investigated. Figure 4.4 shows simulation results for the same 75m radius pond with an inflow comprising two consecutive 1 in 2 year events which occur 12 hours apart. Whilst the pond provides good flow attenuation performance for the first storm event, it fails to meet the 50% reduction on the subsequent storm, only achieving a reduction of 45.2% (The 50% failure threshold is shown by the red line on Figure 4.4). Further simulations with a gradually increasing antecedent period showed that the pond only provided good attenuation for the second storm if the antecedent period was greater than 24 hours.

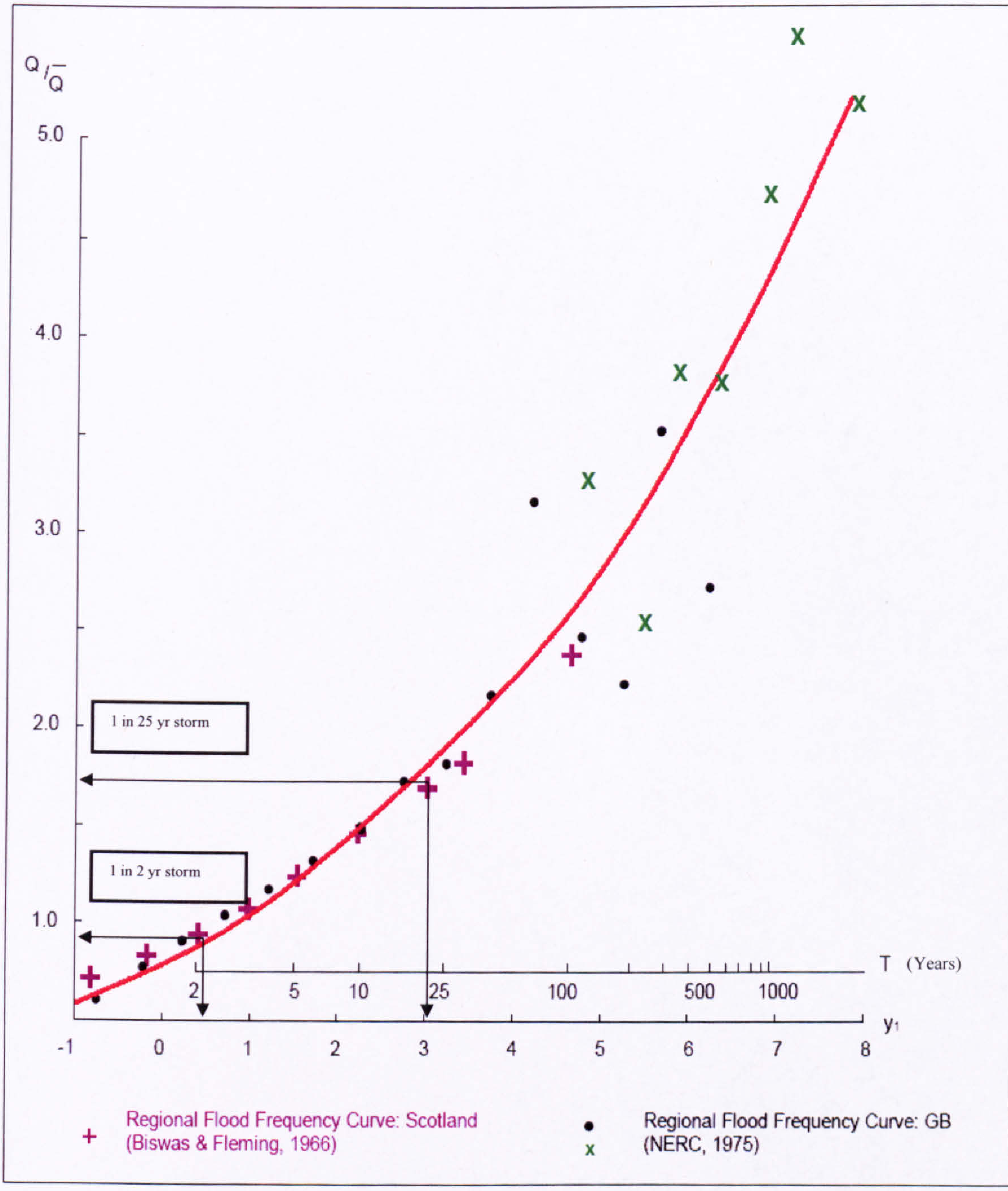


Figure 4.3: Regional flood frequency curves for Great Britain and Scotland (Fleming 2001). See text for explanation of axes and symbols.

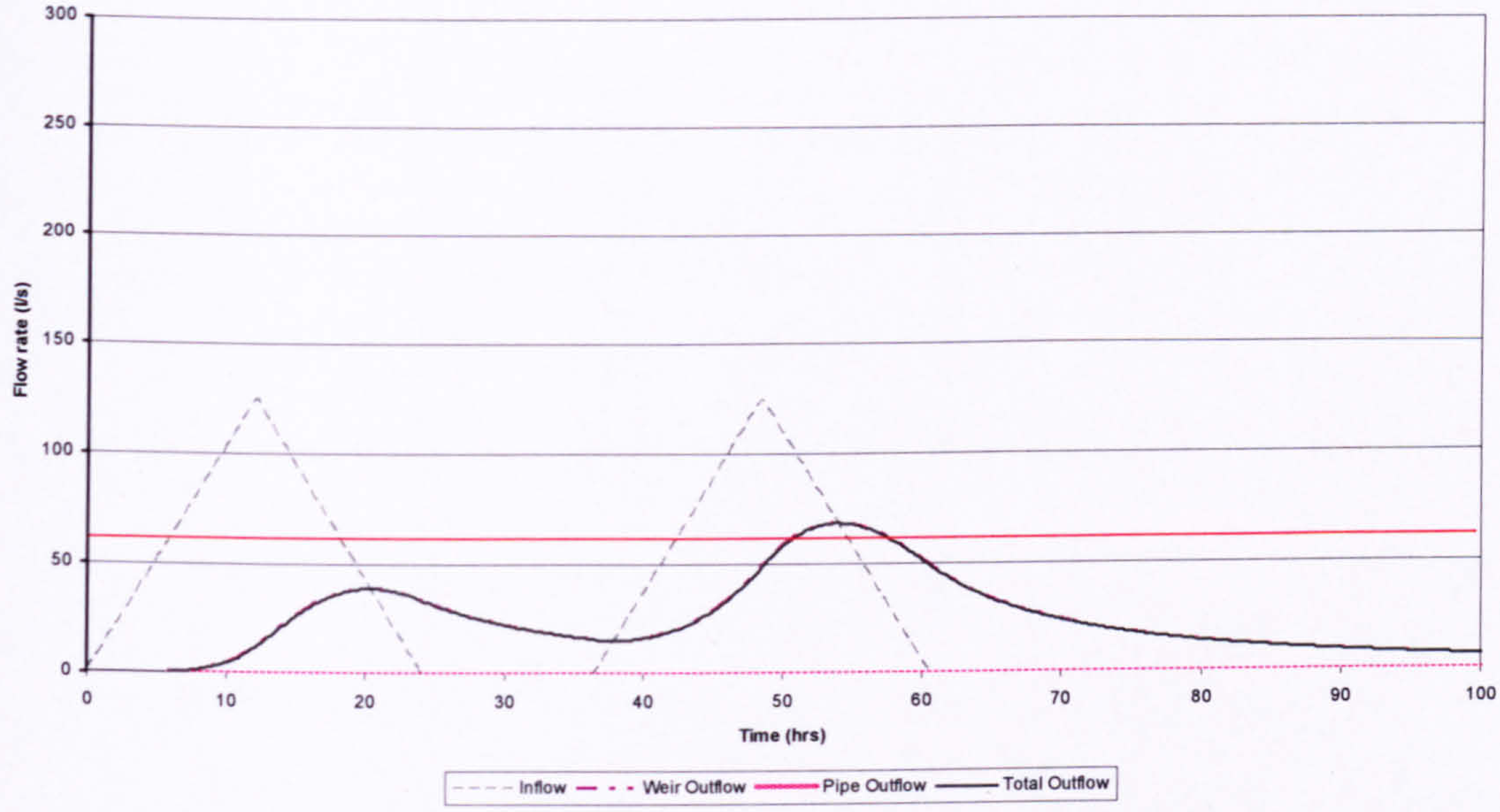


Figure 4.4: Flow hydrograph for two consecutive 1 in 2 year storm events with an antecedent period (of 12 hours), in a 75m radius cylindrical pond

These simulations demonstrate that single event design criteria are insufficient to ensure good flow attenuation of multiple storm events even when they are of a smaller magnitude than the design storm. This is in part due to the single outlet device used here (and typically in Scotland), which does not allow rapid drainage of the pond between storms and provides no temporary storage below the weir crest. In the following section, pond performance using a dual outlet device is explored.

4.2.2 Improving the retention pond outlet configuration

Much of the design literature from other countries [*California BMP Stormwater Handbook*, 2002; *Pitt*, 2005; *Tennessee BMP Guidelines*, 2003] suggests that a multi-stage outlet device can be used to enable flow attenuation of both large infrequent storms and smaller, more frequent events. The simplest form of multi-stage outlet is a dual outlet device such as a v-notch weir together with a pipe at a lower elevation. Simulations with this type of configuration were conducted for a cylindrical retention pond having a 90° v-notch weir at an elevation of 5m (as in the earlier simulations) in combination with an outlet pipe at an elevation of 3m above the base of the pond (i.e. 2m below the weir), modelled as a submerged orifice of diameter 0.1m.

The diameter of the pond was initially adjusted to provide the same 50% reduction of the 1 in 25 year storm event as achieved for the weir only case. Note that with this configuration, it was assumed that the initial water level was equal to the pipe elevation for a single storm event. With the new outlet design, this resulted in a greatly reduced pond radius of 32 m. Such a reduction in pond radius is favourable in terms of both land-take (and construction costs) but it occurs primarily because the dual outlet provides some temporary storage volume in the pond, enabling better attenuation of the design storm. In a pond with a single outlet, however, the only way to achieve a significant improvement in flow attenuation is to enlarge it laterally, by increasing the radius. A further simulation showed that the pond with the dual outlet configuration produced an 82% reduction in the peak flow of a single 1 in 2 year storm event (compared to the 69% reduction achieved using only the weir outlet).

Figure 4.5 shows the simulation results for the pond with the dual outlet device and the same multiple storm inflow used earlier (Figure 4.4), but in this case with no antecedent period between the storm events. Using this outlet configuration enables a much smaller pond to be employed (32m radius compared to 75m radius); furthermore, by comparing Figures 4.4 and 4.5, it can be seen that the pond performance for the second event is much improved when the dual outlet device is used (60% reduction as opposed to 45.2%), even though the length of the antecedent period is now zero. Figure 4.5 shows the individual flows through both outlets, as well as the total outflow. During the first storm event, outflow is via the submerged pipe only, (hence pipe and total outflow overlay each other). It is only during the recession limb of the second inflow event that the weir begins to overflow, and this stops once the water level falls beneath the weir crest.

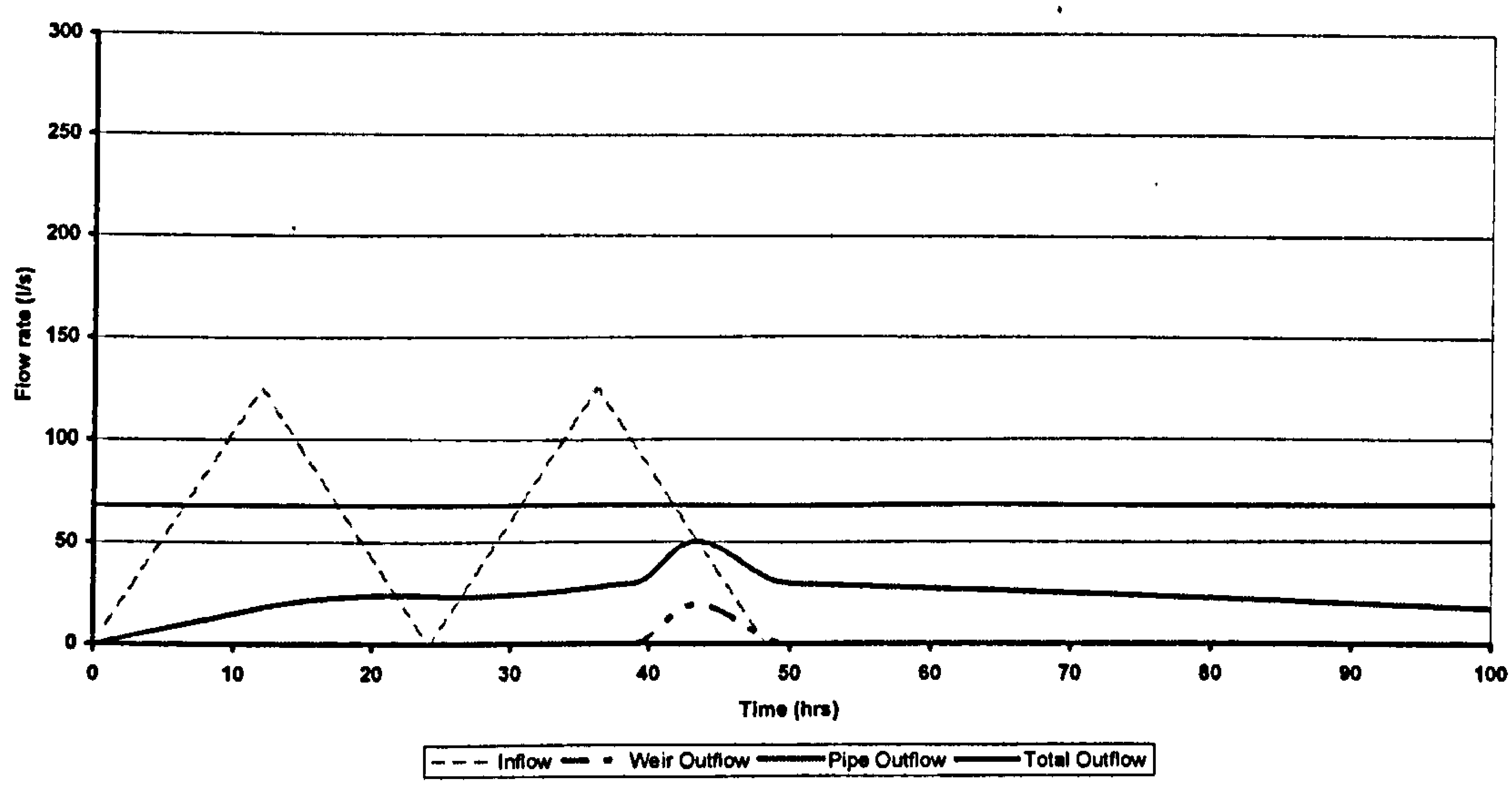


Figure 4.5: Flow hydrograph for two consecutive 1 in 2 year storm events with no antecedent period, in a 32m radius cylindrical pond with dual outlet configuration (weir and pipe). The 50% threshold is indicated by the red line.

4.2.3 Further variations in outlet configuration

The same design approach to that described in the previous section was used to study the sensitivity of these results to variations in the submerged pipe diameter. Table 4.2 (column 2) shows the results of calibrating the pond radius to achieve a 50% flow reduction for the same 1 in 25 year storm event as before, for three pipe diameters. In general, the larger the pipe diameter, the smaller the retention pond required to store the design event, although the reduction in radius is rather modest. By contrast, results for the 1 in 2 year event (column 3), show that percentage peak flow reduction decreases as pipe size increases, reflecting the fact that a significant degradation in pond performance can occur with smaller inflow events when pipe sizes are too large. This occurs because smaller storms are better attenuated by smaller pipes. If too large a pipe is used, there is a tendency for the stormwater to pass straight through the pond without any flow restriction or attenuation. However, if the pipe diameter is too small the pond will not drain fast enough between inflow events. Thus it is important to select an appropriate pipe diameter to enable the attenuation of the range of storms expected. Herein lies a critical design issue. Since both large, infrequent storms and smaller more frequent events occur in the hydrological year, selecting a pipe size to attenuate both sorts of event to meet the pre-defined design standard is crucial in providing good overall pond flow attenuation performance. The effect of pipe size on pond water quality performance is investigated in Chapter 5.

Table 4.1: Peak flow reduction and required antecedent period for various pond configurations and storm events

Pipe diameter (m)	Pond radius required to attenuate the peak flow of the 1 in 25 year storm event by 50% (m)	Peak flow reduction for the 1 in 2 year storm event	Antecedent period required for successful attenuation of consecutive 1 in 25 year storm events	Antecedent period required for successful attenuation of consecutive 1 in 2 year storm events
0.05	33.5	95%	20 x the duration of the design storm	1.5 x the duration of the design storm
0.1	32	79%	6 x the duration of the design storm	No antecedent period
0.15	30	62%	2 x the duration of the design storm	No antecedent period

Table 4.2 also shows the results of simulations conducted to investigate the length of the antecedent period that is required to achieve good flow attenuation for the dual outlet configuration. Two design scenarios were considered for each pond size and pipe diameter combination defined in Table 4.2. In the first scenario, two successive 1 in 2 year storm events were simulated as before, with various antecedent periods in order to find the length of antecedent period required to achieve a 50% reduction in peak flow. The results are given in the final column in the table and show that with the larger pipe diameters, satisfactory performance is achieved even with no antecedent period, but with the smallest pipe diameter some delay between the storms must be present. The penultimate column of Table 4.2 shows results from the second scenario comprising similar simulations but with two consecutive 1 in 25 year events. A reduction in pipe diameter implies that for satisfactory performance there would have to be a longer delay between consecutive inflow events.

If a pond is designed for a known 1 in 25 year single event then to provide equivalent design criteria for multiple events it is necessary to determine the probability of occurrence of multiple design storms and the time period between multiple storms. In this case an analysis of the rainfall series for combinations of storms with antecedent periods below a given threshold would be required. Such an analysis is not currently available for different locations U.K so would have to be performed as part of the pond design process. These simulations demonstrate the importance of determining such multi-event probabilities.

4.3 Investigation of flow attenuation in Linburn pond

In the following section, the knowledge gained from the simulations discussed previously in the chapter is used to investigate the performance of an existing retention pond in Scotland and to propose design modifications for it under both current and future climate scenarios.

4.3.1 An Introduction to Linburn Pond Catchment

Figure 4.6: Map of Scotland showing Dunfermline (the location of the DEX development), and the city of Edinburgh for reference.

The Dunfermline Eastern Expansion (DEX) area is an area of rapid urban development 28km north of Edinburgh in Eastern Scotland (Figure 4.6). The 5.5km^2 area was predominantly comprised originally of greenfield land underlain by a low permeability clay soil. Construction of the site began in 1997, with SUDS in place and established by 1999. Development of the DEX site was expected to continue until 2020. The expansion proposal included a leisure park, residential areas with community facilities, including primary schools, as well as industrial and commercial developments such as supermarkets. The plans included the provision of open parkland and woodland areas [*Hutton*, 1997]. The natural surface water drainage from the site discharges into four burns: the Calais Burn, Linburn, Keithing Burn and Pinkerton Burn – all of which flow through substantially built-up areas down stream of DEX. SUDS were therefore made a planning condition at the DEX development site. This was primarily due to concerns over degradation of the quality of the water being discharged to local rivers. However, the catchments further downstream of the site had an existing flooding problem, and it became clear from the planning proposals that without on-site attenuation, the development of the DEX complex would significantly increase the probability of flooding in the heavily built-up areas downstream [*CIRIA*, 2001; *Hutton*, 1997].

DEX is the first site of its size and complexity in the UK to use sustainable drainage across the entire development area. Systems used include retention basins, swales, permeable paving, regional extended detention ponds and wetlands. The ponds and basins used on the site have been designed to attenuate the runoff that can be expected from up to 90% of storms occurring in a single year [*CIRIA*, 2001].

Linburn Pond is one of several ponds constructed at DEX in 1998 and it was chosen as a case study for simulations in this thesis since design information and monitoring data were available due to a larger-scale project that monitored performance of several SUDS at DEX from 1999-2003. Its catchment area is 67.5ha and drainage within the catchment is from east to west with maximum slopes of 10%. Land use was predominantly grassland, however, the initiation of the DEX development has resulted in the construction of medium density housing in the north and northwest of the catchment with a highway running from east to west. Figure 4.7 shows the DEX site with the area that drains to Linburn pond delineated. Linburn Pond receives incoming stormwater runoff through a complex pipe network that drains the nearby roads, a residential housing estate, a commercial leisure park complex comprising, a construction site and the surrounding undeveloped grassland.

Linburn Pond also drains 6 detention basins situated upstream in the catchment which collect runoff entirely from roads and highways [*Spitzer*, 2001]. Most of the stormwater enters the pond through an inlet at its eastern end (there are three other minor inlets). The outlet device comprises four 90° v-notch weirs all with the same weir crest elevation. Some flow monitoring data for the pond outlet from 2000-2001 was available for the study, and at the time of monitoring, the catchment had a built-up surface area of approximately 10%.

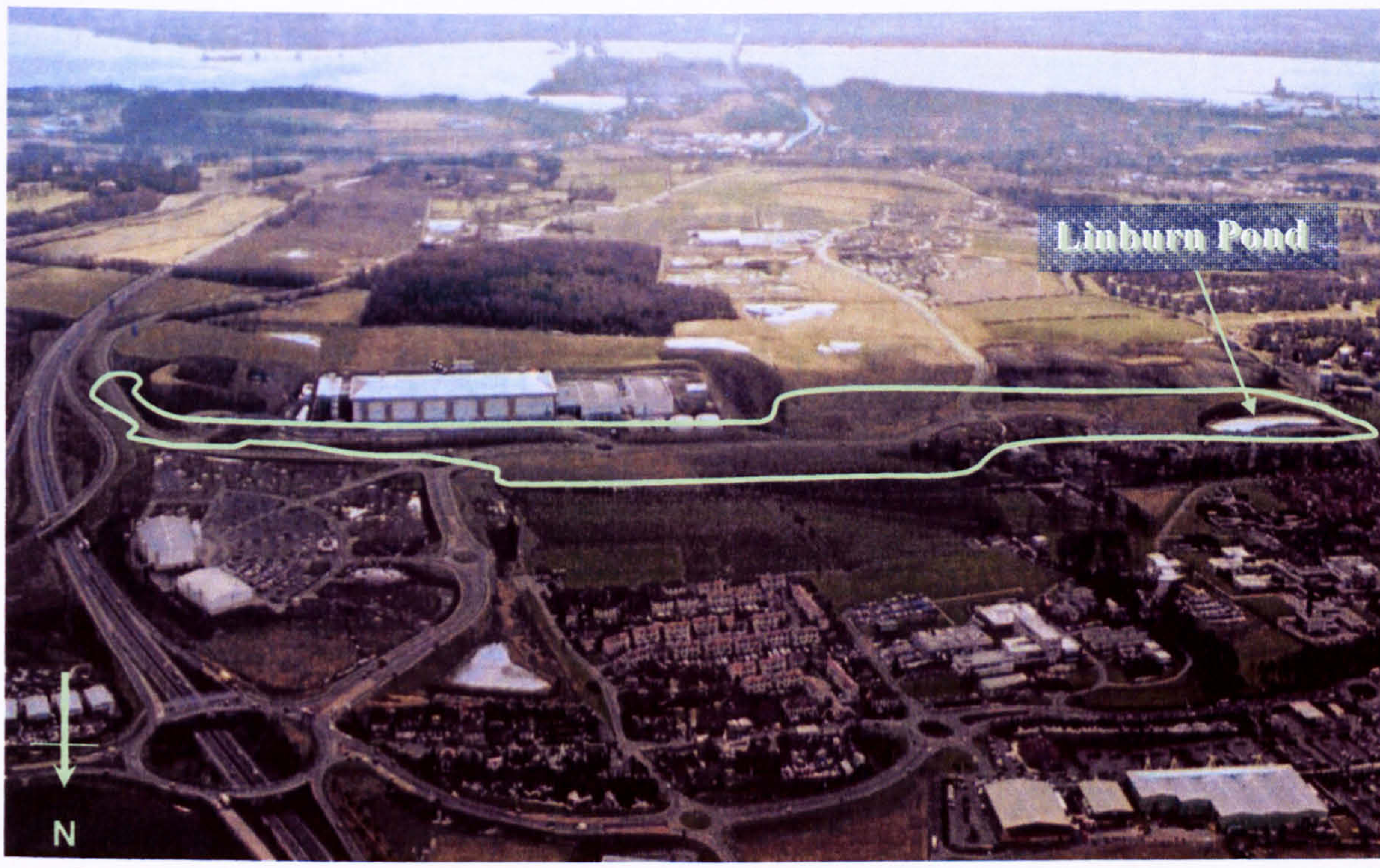


Figure 4.7: Aerial photograph of the Linburn Catchment (outlined in green) within the DEX development site, Dunfermline, Scotland 2002 (Source: Abertay University).

4.3.2 Derivation of an inflow series for Linburn pond

Although pond outflow data for the period May 2000 to May 2001 were available from Abertay University for Linburn Pond inflow was not monitored. Consequently, it was necessary to construct an inflow record for the period May 2000 to May 2001 using data from the nearest rain gauge at South Fod, located to the north of the pond within the Linburn catchment. The South Fod gauge was monitored by Adolf Spitzer, a research student from the Urban Water Technology Centre, Abertay University, Dundee during this period. Data collected over the sampling period was compared with data from a nearby rain gauge, the Annfield gauge, which is part of a hydrometric network monitored by SEPA.

According to [*MUDWADE*, 2000], data from the gauge compared favourably both in terms of annual totals and individual storms Records from the South Fod gauge, a 0.2mm tipping bucket rainfall gauge were used to synthesise an inflow series using the simple relationship shown below [*Shaw*, 1997]

$$Q_i = Q_d C A \qquad (4.1)$$

where, Q_i is the synthesised inflow, Q_d is the depth of rain, falling in a 15 minute period, C is a coefficient describing catchment imperviousness and A is the area of the catchment (km^2). The synthesised inflow is shown with the observed outflow for 2000-2001 in Figure 4.8. The inflow was generated from equation (4.1) by assuming a direct runoff contribution of 10% to represent the percentage of the catchment area under development during the period of outflow monitoring. It was then considered that this developed area of the catchment was completely impervious and the remainder did not contribute to pond inflow. The resulting inflow hydrograph is shown in Figure 4.8. Also plotted on Figure 4.8 is the observed outflow at the pond outlet. A 10 day period of the synthesised inflow and measured outflow for October 2000 shows that the inflow is greatly exceeded by the observed outflow from the pond. This is even clearer in the detailed plot for October shown in Figure 4.9. In fact, an analysis of the annual water budget determined by comparing total annual rainfall with total annual pond outflow suggests that, for the year 2000-2001, outflow was approximately 10 times greater than the maximum possible inflow (i.e. assuming all incoming rainfall over the whole catchment surface is immediately transferred via drainage networks to the pond).

Furthermore, the data demonstrate two anomalies in the observed outflow from the pond. Firstly, examination of the outflow data showed that outflow from the pond is continuous all year round and never falls to zero (analysis of the complete outflow dataset showed that even during the dry inter-event periods there was an average outflow of 16 l/s). Secondly, for individual storm events, such as those shown in Figure 4.9, a distinct recession limb is visible on the outflow hydrograph. Clearly therefore, a large volume of water (at least 90% of the outflow) is derived from another source. Visual analysis of Figure 4.8 also demonstrates greater outflow between rainfall events in winter compared to summer and a significant recession following rainfall events. Such anomalies are typical of a significant contribution to the outflow from groundwater seepage. In fact, to match the volume of observed outflow, the pond must be draining groundwater from an area approximately 10 times greater than the 67.5ha surface water drainage catchment.

To enable the construction of a realistic inflow to the pond, a direct groundwater contribution was added to the rainfall-runoff. The groundwater source was modelled as a simple linear reservoir [*Butsaert and Niber*, 1977]. Using a similar storage routing approach to that which underpins the pond flow model described earlier, the governing equation is:

$$\frac{dS}{dt} = Qr - Qg \qquad (4.2)$$

where $S\ (= aQ_g^{\,b})$ is the groundwater reservoir storage, Qr is the recharge (from infiltrated rainfall), Q_g is the groundwater outflow and a and b are empirical coefficients. In calibrating the equation a and b were adjusted so that (i) the inflows to the pond showed the characteristics of groundwater influence that were clearly missing from the direct rainfall-runoff and (ii) the simulated volumes of inflow to, and outflow from, the pond were approximately equal. This resulted in around 10% of the inflow being direct runoff and 90% of the inflow entering the pond via groundwater. A satisfactory simulation (i.e. one in which the synthesised inflow was consistent with the observed outflow) of the baseflow recession on the outflow hydrograph was achieved with values for a = 0.002 and b = 1.1. For comparison, the new simulated inflow to Linburn Pond including the groundwater contribution (referred to here as the Predicted Inflow), for the same 10 day period as before, is shown alongside the observed pond outflow in Figure 4.10.

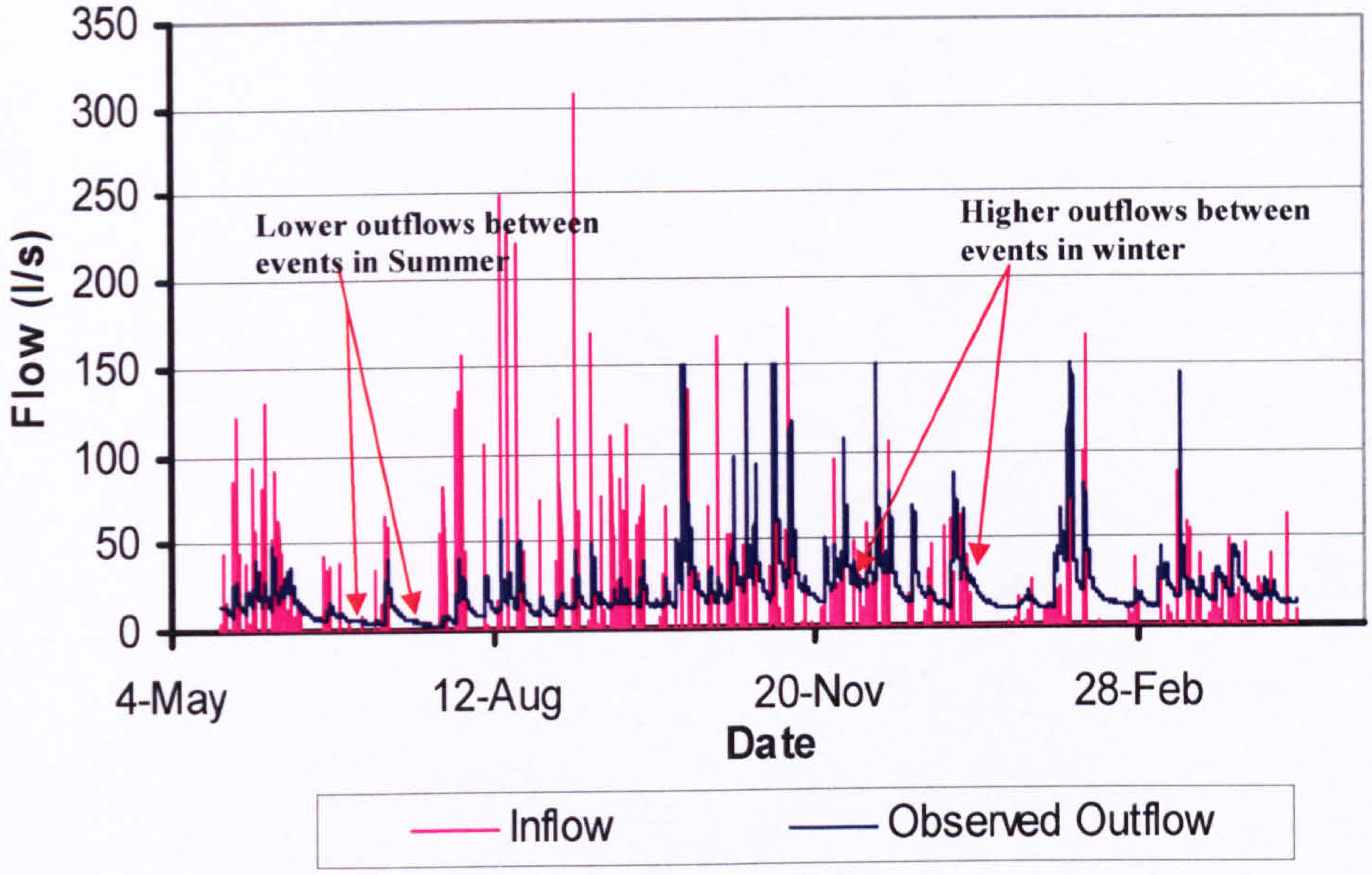


Figure 4.8: Simulated Inflow and Observed Outflow at Linburn Pond 2000-2001

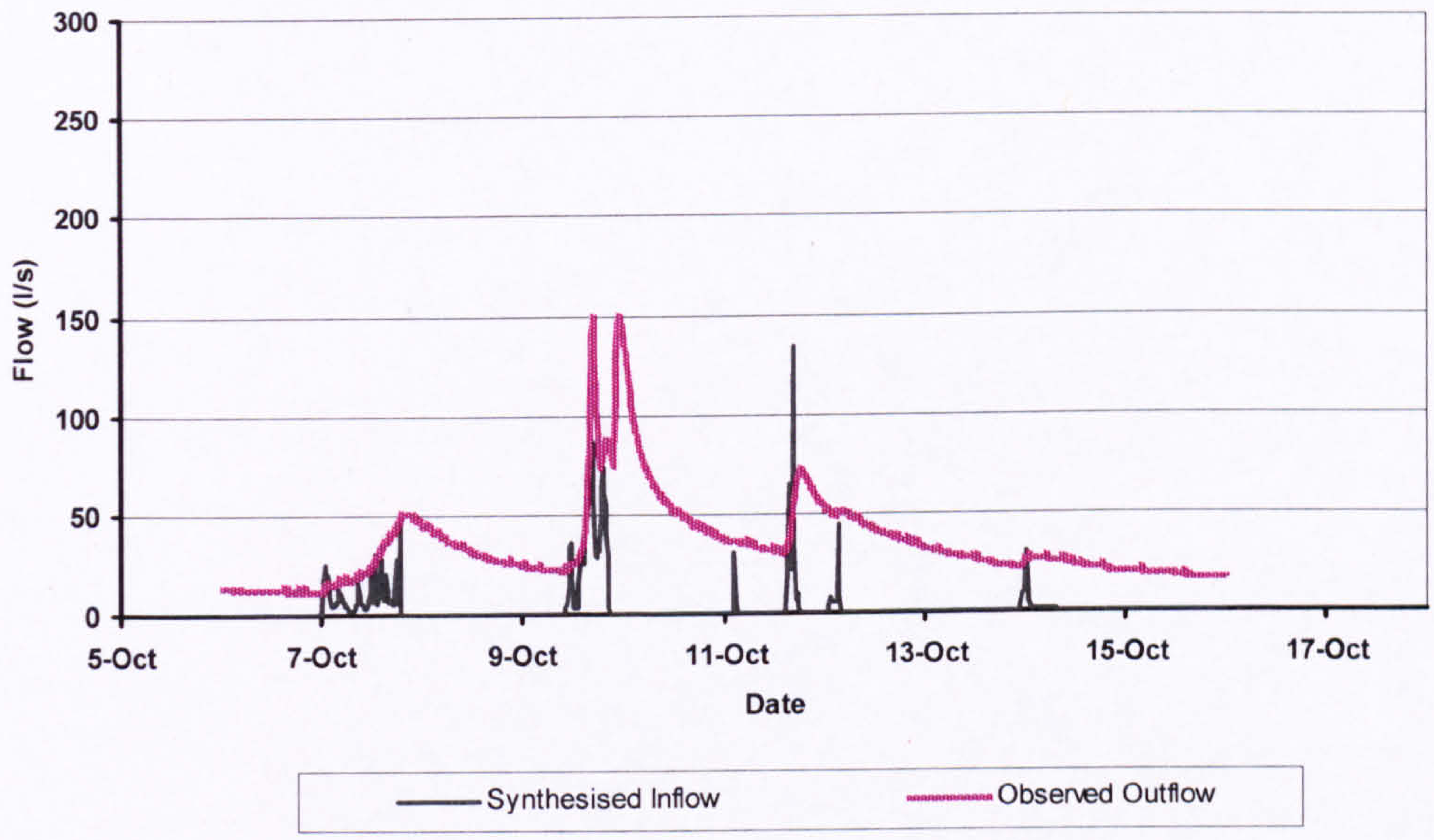


Figure 4.9: Synthesised Inflow and Observed Outflow at Linburn Pond, 6-16th October 2000

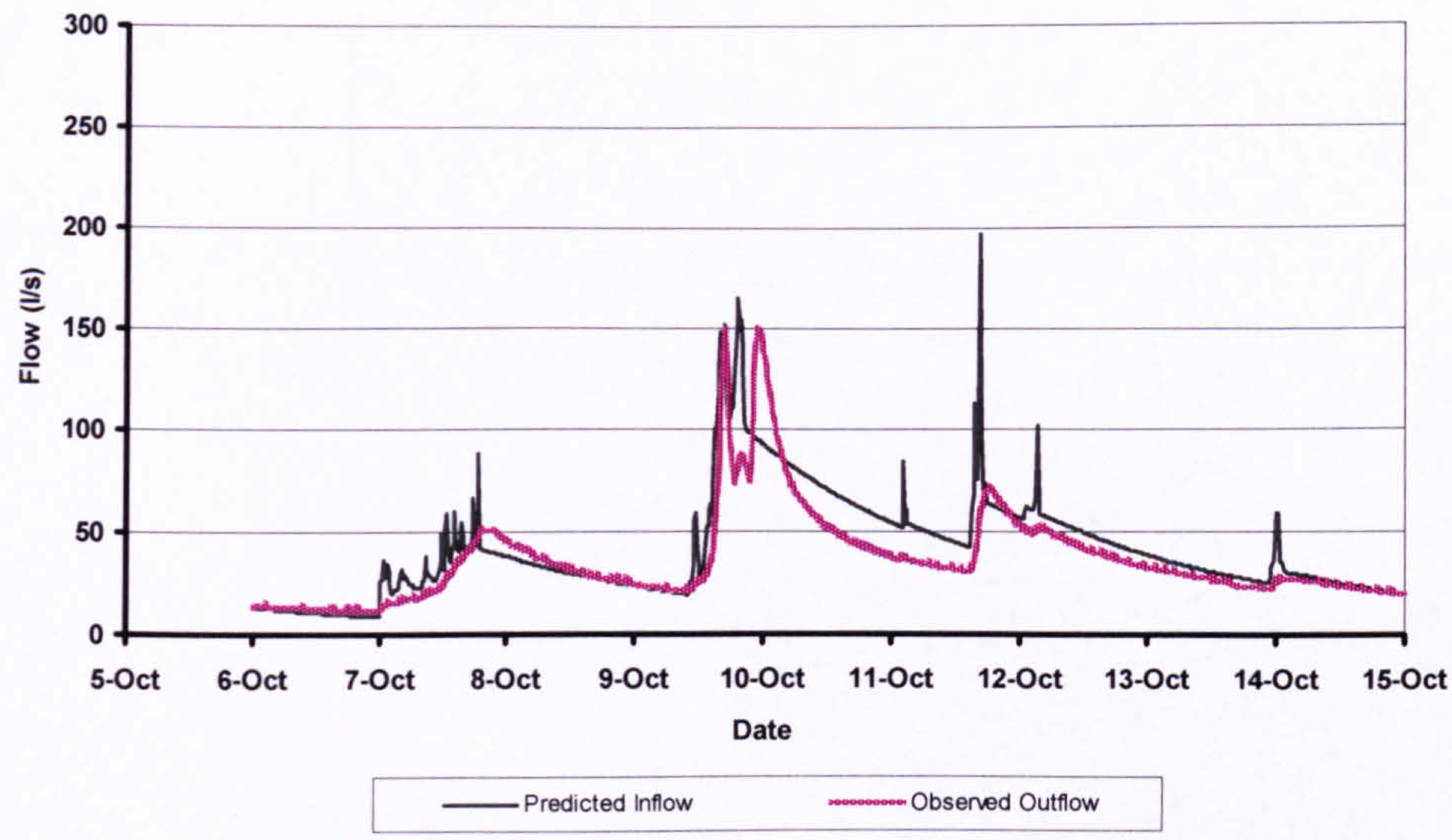


Figure 4.10: Predicted Inflow and Observed Outflow at Linburn Pond, October 2000

Once a suitable inflow record for Linburn Pond had been generated it was then used with the pond model to simulate the outflow. The flow model was configured to simulate the current conditions at Linburn. Linburn Pond is approximately 'kidney-shaped', which for the purposes of modelling, was translated into a cylinder of the same depth and surface area. This resulted in a pond with a volume of about 15,000m^3 and a radius of 40m. The weir crest elevations of the four 90° v-notch weirs are located at 2.8m above the base of the pond based on the design drawings. Figure 4.11 shows the simulated outflow from the model based on the predicted inflow (using the groundwater algorithm to synthesise the inflow) and the observed outflow for Linburn Pond for two time periods during 2000-2001. From these figures it can be seen that the model produces an outflow that is a good match to the observed data, however in general, the model tends to slightly over predict both the storm peak flows and recession curves. Since this over prediction includes the recession curve, it is most likely to be due to the simplistic approach used to calculate the groundwater inflow to the pond. However, since no inflow data are available for Linburn Pond that might enable the groundwater contribution to be better simulated, a more complex approach is unwarranted. Furthermore, since it is 'the pond design' itself that is of interest here, not the precision of the inflow prediction, any simulation errors in the inflows should not compromise comparisons of different pond designs.

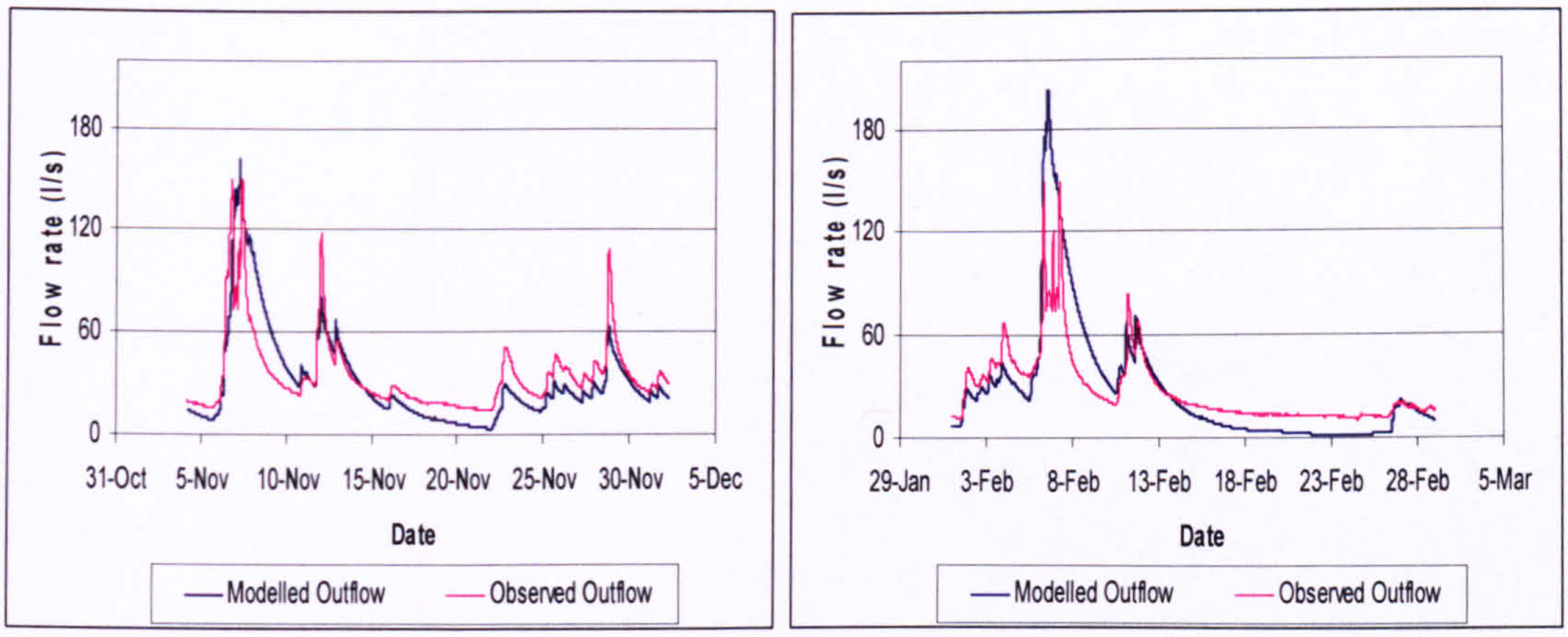


Figure 4.11: Model Outflow and Observed Outflow at Linburn Pond for different time periods, 2000-2001

An analysis of the above simulations in terms of pond performance can be achieved by considering the peak flow reduction for the high flow events in Figure 4.8. In terms of pond performance, Figure 4.12 demonstrates that peak flows are not greatly attenuated (with the exception of the storm on 12th Oct) and, further, there is very little time lag between peak inflow and peak outflow. In its current configuration, therefore, it is clear that the pond's performance is very poor, with good attenuation not even being achieved for the smaller events shown in Figure 4.12. This is consistent with the pond having no temporary storage volume because it is continually overflowing over the weir.

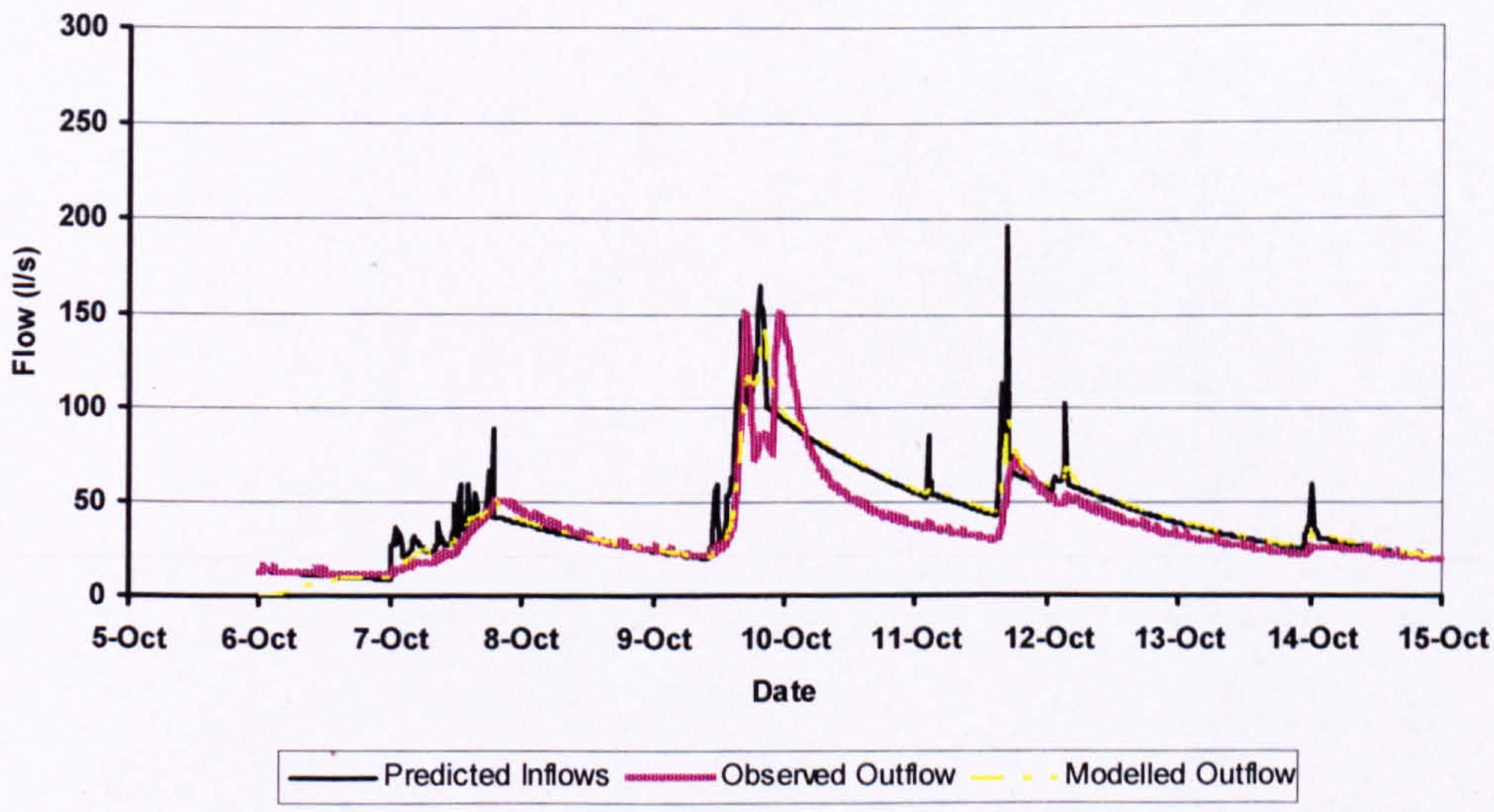


Figure 4.12: Predicted Inflow, Observed Outflow and Modelled Outflow at Linburn Pond, October 2000

4.3.3 Initial design recommendations

In this section, the extent to which the performance of Linburn Pond could be improved by incorporating two design changes is investigated. The first adaptation to the design considers the use of an effective impermeable liner to prevent groundwater inflow. The second adaptation is the installation of a dual outlet device, based upon the evidence presented in section 4.1 of the benefits of such a system.

By preventing the dominant groundwater component of the inflow from reaching the pond it is highly likely that flow attenuation performance would be improved. One way to achieve this would be to make the pond impermeable to the local groundwater table by lining it with an impermeable membrane. To quantify the effect of this, simulations were conducted with the pond flow model assuming a reduction of 90% in the groundwater contribution to the inflow, the remaining 10% of the groundwater contribution being assumed to still reach the pond via agricultural field drains which drain into inflows upstream of the pond. A comparison of the inflows and outflows for the 2000-2001 monitoring period for unlined and lined pond configurations is shown in Figure 4.13. A quantitative assessment of the improvement in performance achieved by lining the pond was made using the failure criteria discussed earlier, wherein good flow attenuation is achieved when a pond reduces peak outflow by at least 50%. Using this criterion, the unlined pond failed 18 times out of a potential 73 storm events, whereas the lined pond fails only twice during the same period. Although this method provides and excellent tool for assessing the effect of design on pond performance, it may not provide an insight into the effect of failure on the catchment. A more suitable technique might be one which considers the degree to which the pond fails. Such a technique would provide a number of different thresholds each with a different level of risk to the catchment. For example a minimum threshold failure may not cause any great effect on the catchment, where as a higher threshold might indicate that sustained high flows in the pond and in the surrounding catmint may pose a significant risk to aquatic life and habitats. A very high failure, however, may indicate that the catchment and receiving waters may be at high risk of flooding.

Since the thesis merely considers the effect of changing pond designs on performance, the simple method used previously to determine pond failure is justified.

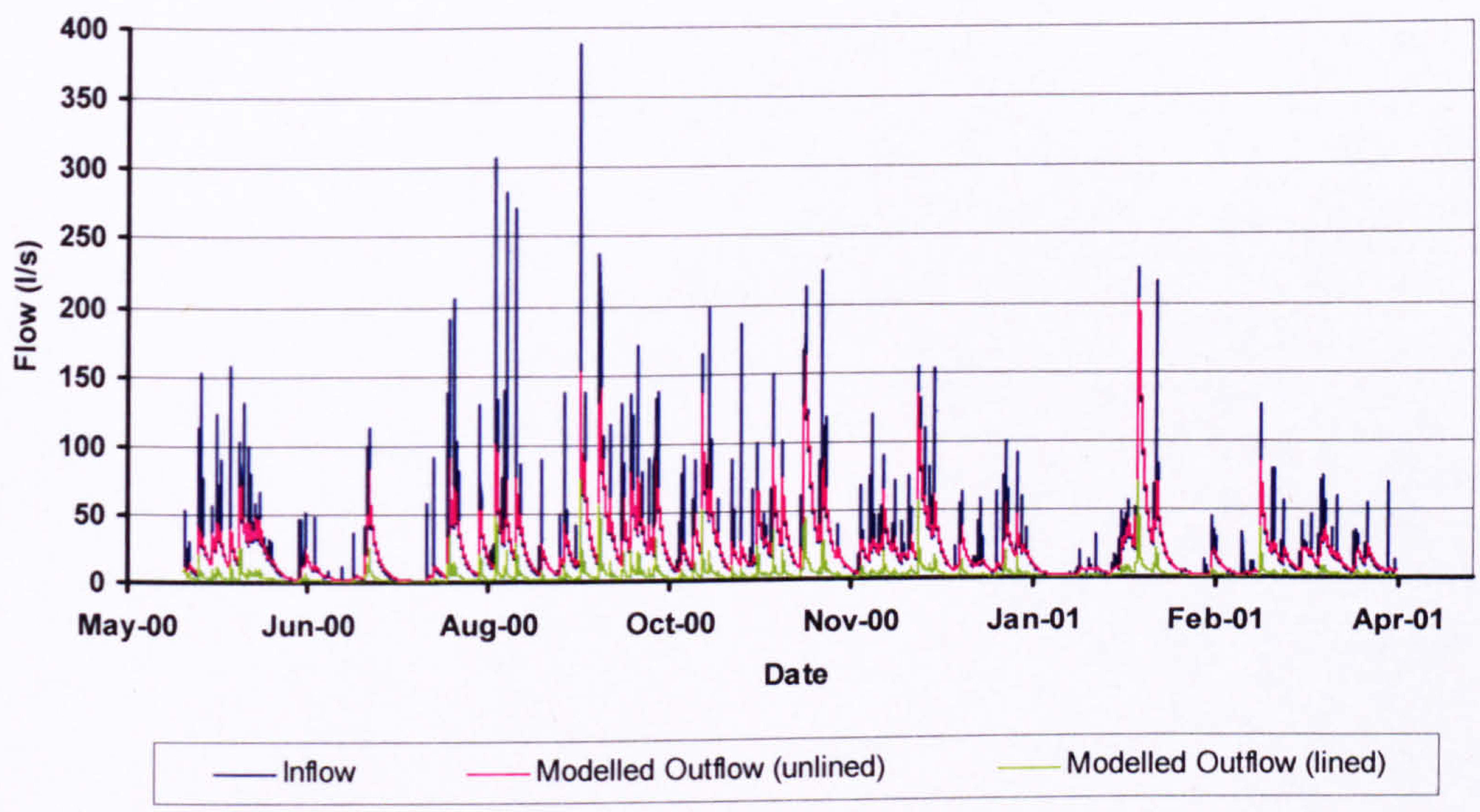


Figure 4.13: Inflow and Modelled Outflow for Linburn Pond (lined and unlined configurations)

Since Linburn Pond is an existing retention pond with well established vegetation and wildlife, it would be difficult to line the pond retrospectively without interfering with the pond ecosystem. Therefore, an alternative approach for improving pond performance is investigated whereby a multi-level outlet configuration is considered. Simulations of the *unlined* Linburn Pond with a dual outlet configuration, combining the four v-notch weirs with a submerged pipe, were undertaken to investigate the independent effect of providing some temporary storage beneath the weir crests. Figure 4.14 shows results from simulations using a 0.13m diameter pipe located 1.8m above the base of the pond (i.e. 1m below the weir crests), for the same 10 day period in October 2000 shown in Figures 4.9, 4.10 and 4.12. During the first two storms the combined outflow from the pond is reduced compared to the single outlet device case shown in Figure 4.12. For example, the peak outflow for the second event is about 110 l/s compared to 150 l/s for the weir only case. The explanation for this improvement is apparent from the individual weir and pipe flows

(Figure 4.14). With the introduction of the submerged pipe, flow over the weir does not now occur until after the onset of the **second** inflow peak. Prior to this outflow is via the pipe only. During the period when both the weir and the pipe are in operation, however, the pond performance is little better than before.

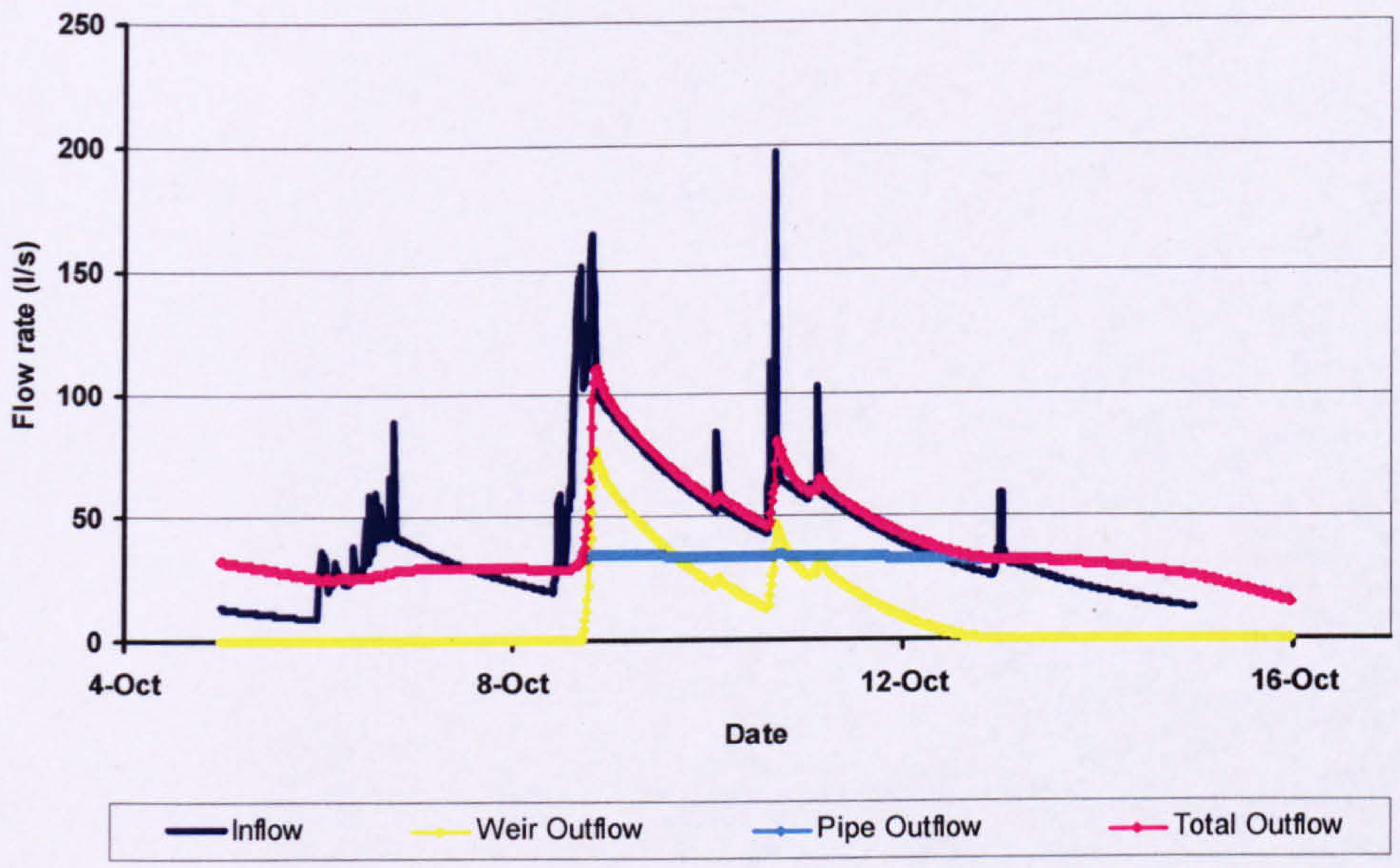


Figure 4.14: Inflow and Modelled Outflow at Linburn Pond for a storm sequence in October 2000

4.4 Further design recommendations for Linburn Pond incorporating climate change

In the previous section it was demonstrated that lining the pond or providing some temporary storage volume would improve its flow attenuation performance. Unfortunately, the former would lead to the destruction of the pond's existing established ecological communities, and so this is not a viable recommendation. Further improvements, therefore, need to consider increasing the size of the temporary storage volume by incorporating a dual outlet device as discussed above. However the provision of this volume is limited by guidelines for the minimum pond water depth. [*CIRIA*, 1993] estimate the minimum water depth required to maintain a permanent pool of ecological value to be 1m, which restricts the difference between the pipe and weir elevations in Linburn pond to a maximum 1.8m. Compared to the configuration used in the simulation shown in Figure 4.14, therefore, an extra 0.8m depth could be used for extra temporary storage between the two outlets. Any greater temporary storage volume would require an increase in the current pond surface area also.

4.4.1 Choice of Storm Event Magnitude and Frequency for Linburn Pond

To propose further improvements to the design of Linburn Pond that would be relevant to a typical design life of, say, 25 years, the issues of temporary storage, event magnitude and event sequences have been combined with some climate change considerations. In order to do this, it was necessary to determine the magnitudes of the 1 in 25 year and the 1 in 2 year flow events for the Linburn catchment. The flood frequency curves for Great Britain (Figure 4.3) suggest that the peak flow of the 1 in 25-year event is approximately double that of the 1 in 2-year event. To aid the quantification of the 1 in 25 year storm event for Linburn catchment, a 38-year daily rainfall data set from The British Atmospheric Data Centre (BADC) for the Tullyallan gauge, a rain gauge within 23km of the catchment was used, primarily because a long enough record of higher resolution data (from South Fod) was not available.

Previous simulations were conducted using an isosceles triangular inflow hydrograph, however this section uses a real storm hydrograph to incorporate storm characteristics such as shape, recession etc. The largest event at a 15-minute resolution in the Linburn (South Fod) data set was 41.5mm. Although a comparison with the annual maximum 5-day Tullyallan (Figure 4.16) shows this to be closer to a 1 in 1 year event than a 1 in 2year event, double this value (82 mm), however, is similar to that of a 1 in 25 year event at Tullyallan, where only one event significantly exceeded 82mm in 38 years. Therefore, this event was taken to represent a worst case scenario for Linburn Pond.

Consequently, in keeping with the relationship between the 1in 2 year and the 1 in 25 year storm used earlier in this chapter and for the purpose of pond design, the event highlighted in Figure 4.15 (of 41.5mm) is assumed to be the 1 in 2 year 5-day rainfall at Linburn Pond, with a corresponding 1 in 25 year event being 83mm (double the 1 in 2 year event). It is recognised that in reality, the relationship between rainfall and runoff is much more complex than assumed here, and that a doubling of rainfall would not necessarily lead to a doubling of flow. However for the purpose of these simulations, an estimate of the 1 in 25 year runoff (i.e. flow into the pond) was required. In the absence of a complete or comprehensive data set to provide this estimate, this simple method is justified.

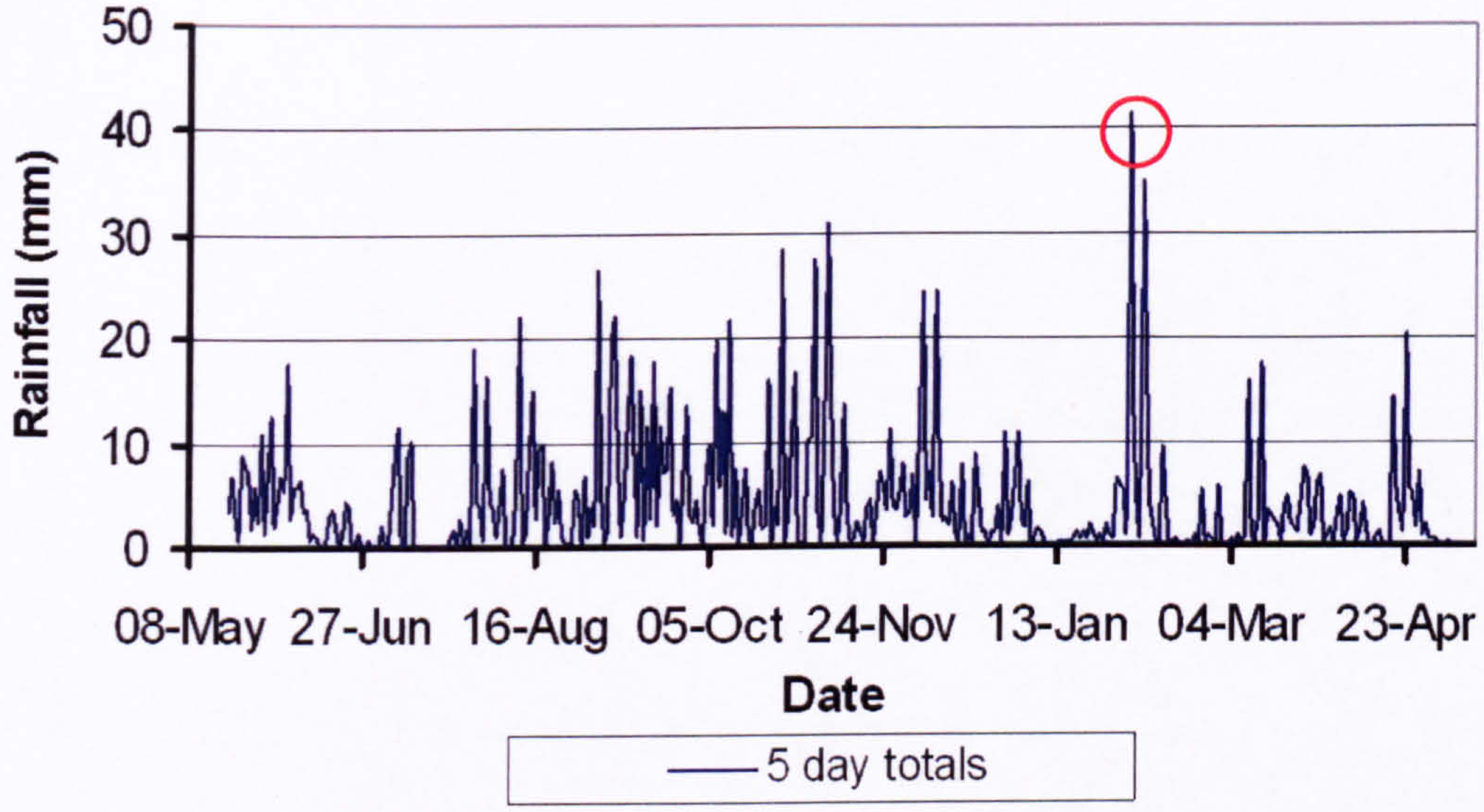


Figure 4.15: The largest 5-day storm at the South Fod Gauge (Linburn), May 2000-2001

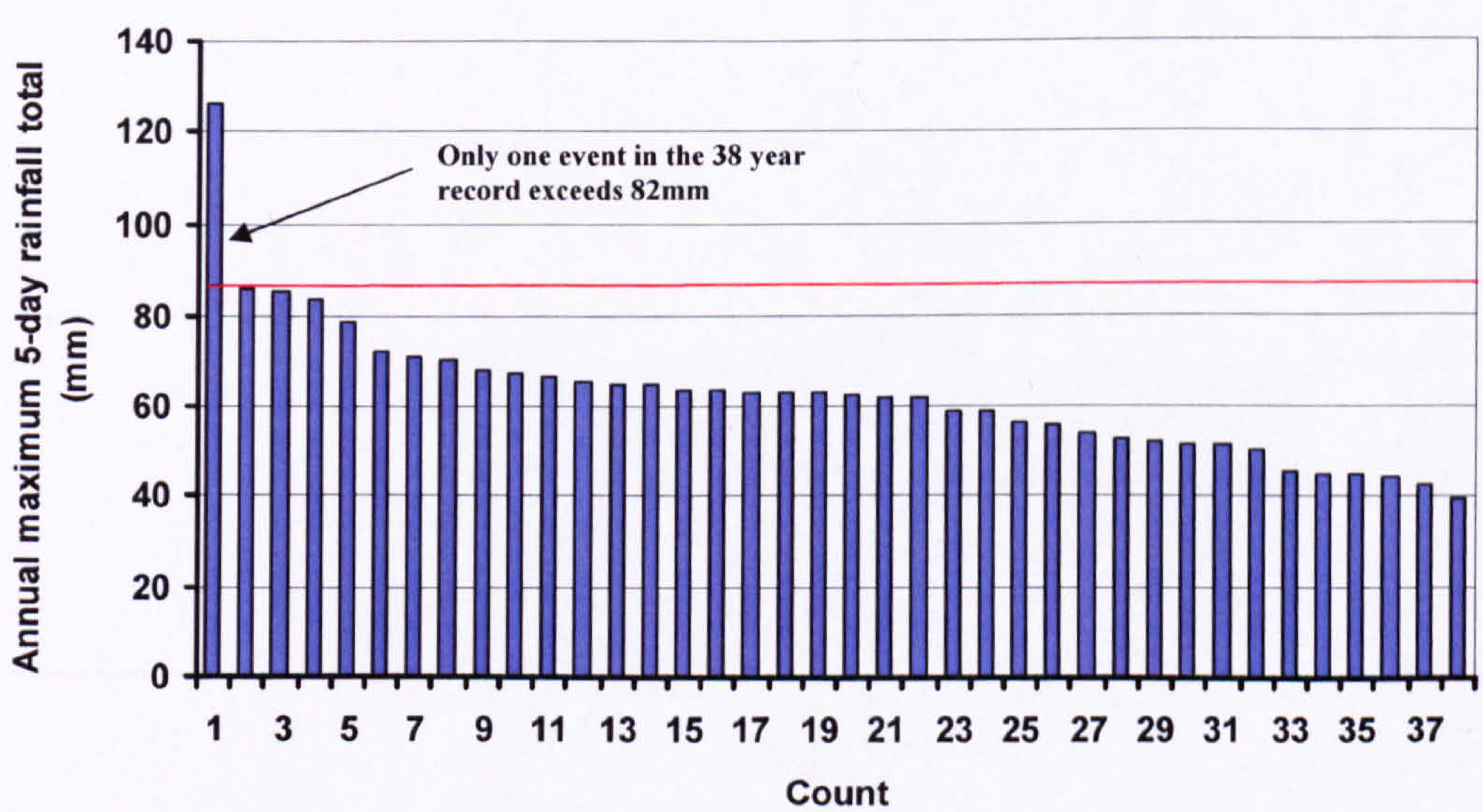


Figure 4.16: Plot of Annual maximum 5-day rainfall events in ascending order at Tullyallan

Figure 4.17 shows the year long hydrograph produced from the predicted inflows and the modelled outflows for Linburn Pond from the rainfall series shown in Figure 4.15 and highlights two large inflow events to the pond which occur in September 2000 and February 2001, respectively. Although the storms have the same duration of 5 days, model simulations using the current pond configuration (i.e. unlined with a single level outlet) showed that, the smaller event in February 2001 posed a greater challenge to pond performance, in terms of peak outflow attenuation, than the larger event in September 2000. Hence, the February 2001 event is used in the following sub-section to re-design Linburn pond to provide adequate flow attenuation throughout its design life.

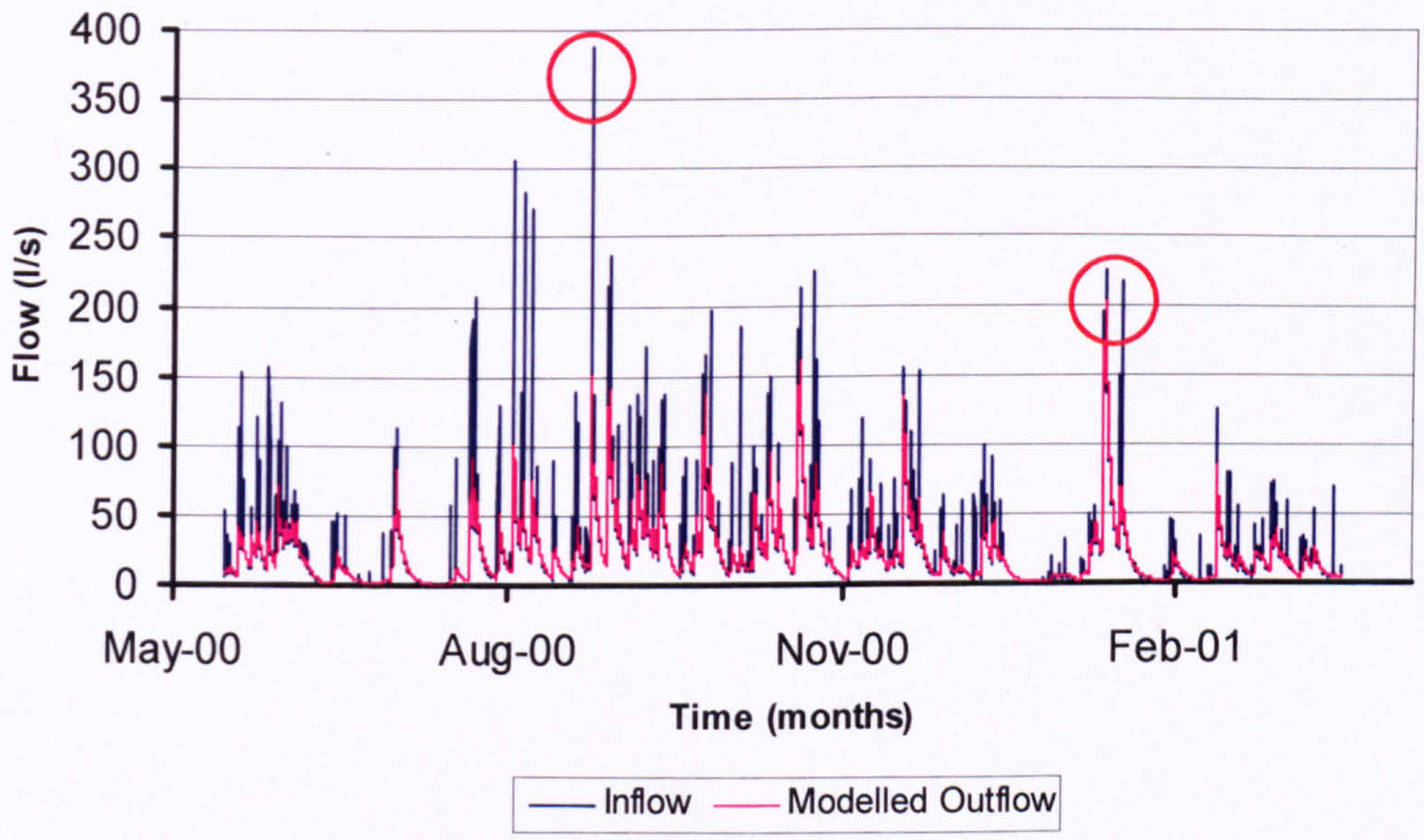


Figure 4.17: Predicted inflow and modelled outflow (using 15 minute data from South Fod guage), May 2000-April 2001.

4.5 Climate change issues

Global warming, as a consequence of increased atmospheric concentrations of greenhouse gases is expected to produce an increase in the frequency and magnitude of extreme hydrological events, both world-wide and in the UK [*IPCC*, 1996; *The Foresight Future Flooding Report*, 2004]. Recent Global Climate Model (GCM) projections have predicted increases in the frequency and intensity of heavy rainfall at northern latitudes [*Ekstrom et al.*, 2004] which are consistent with observations of increased rainfall intensity in the UK over the past 20 years [*Fowler and Kilsby*, 2003].

Simulations undertaken by Ekstrom *et al.* (2004) using a Regional Climate Model (RCM), HadRM2H, suggest there will be an increase of up to 10% in rainfall event magnitudes in all regions of the UK for low return periods (5-10 years). For higher return periods (>25 years), the increases are smaller across central and southern parts of the UK and there are even reductions in some regions of England. However, at higher return periods in the north and west, the relative increase in event magnitude is much greater, with the largest being estimated for East Scotland, the location of Linburn Pond. Here, increases in event magnitude of 16% are predicted for the 5-day duration 1 in 25 year event and 20% for the 5-day duration 1 in 50 year event.

Clearly the results from Ekstrom *et al.* (2004) have serious implications for the future performance of retention ponds in the UK. Consequently, to redesign Linburn Pond for successful flow attenuation over a 25-year operating period, it is necessary to incorporate the predicted increase in the magnitude of the 1 in 25 year event over this time span. Since no published data for the expected increase in event magnitudes exist for the next 25 years, the 100 year predicted increase (Ekstrom *et al.*, 2004) was taken as erring on the side of caution. Figure 4.18 shows simulated inflow and outflow for Linburn Pond under conditions of climatic change. The 1 in 25 year inflow was derived by doubling the magnitude of the 1 in 2 year inflow event identified in the previous sub-section and then further increasing it by a factor of 16% to account for climate change (see above). In Figure 4.18 a peak flow reduction of 50% of the inflow was achieved using a 0.15m pipe diameter at the lowest possible elevation (1.8m below the weir crests) and by increasing the pond radius to 78m. In the absence of climate change, simulations showed that a pond radius of 71m was sufficient for the same dual outlet configuration. Currently Linburn pond has a radius of 40m, however, the simulations above have demonstrated the pond's inability to cope with increases in rainfall magnitude. In practical terms increasing the size of a pond, such as Linburn, to accommodate future changes is not feasible, financially or in terms of situation, since the land around the pond has been further developed with housing and commercial property. However, it is certain that runoff in the Linburn catchment will continue to increase, if not through changes in climate, runoff will certainly increase due to continued urbanisation of the catchment.

Figure 4.19 shows the performance of Linburn Pond for two consecutive 1 in 2 year events with the 78m radius redesigned pond and with the existing design. Again, the 1 in 2 year events have been adjusted for climate change (this time the event magnitude has been increased by 9%, using the closest available predictions for a 1 in 5 year 5-day duration event from Ekstrom *et al.* (2004) as the closest information available). The first event is attenuated very well in the re-designed pond, being largely stored below the weir crest level, whereas the second event is rather poorly attenuated (peak flow reduction of 37%) since there is little storage volume left. Further simulations showed that the required 50% reduction for the second event would be achieved by the new pond only if there were at least 6 days antecedent period between any two 1 in 2 year events. In contrast, the existing pond design (i.e. no submerged pipe and a 40m radius) performs poorly, no matter what the antecedent period, attenuating the first and second event by only a few percent. Adhering to current planning restraints for pond depth (which can be no greater than 3m), to reduce the size of the pond required in Linburn catchment would require the use of a dual-level outlet and a pond liner.

Figure 4.18: Simulations of the 1 in 25 year inflow event in Linburn Pond under possible climate change scenario (16% magnitude increase in inflow)

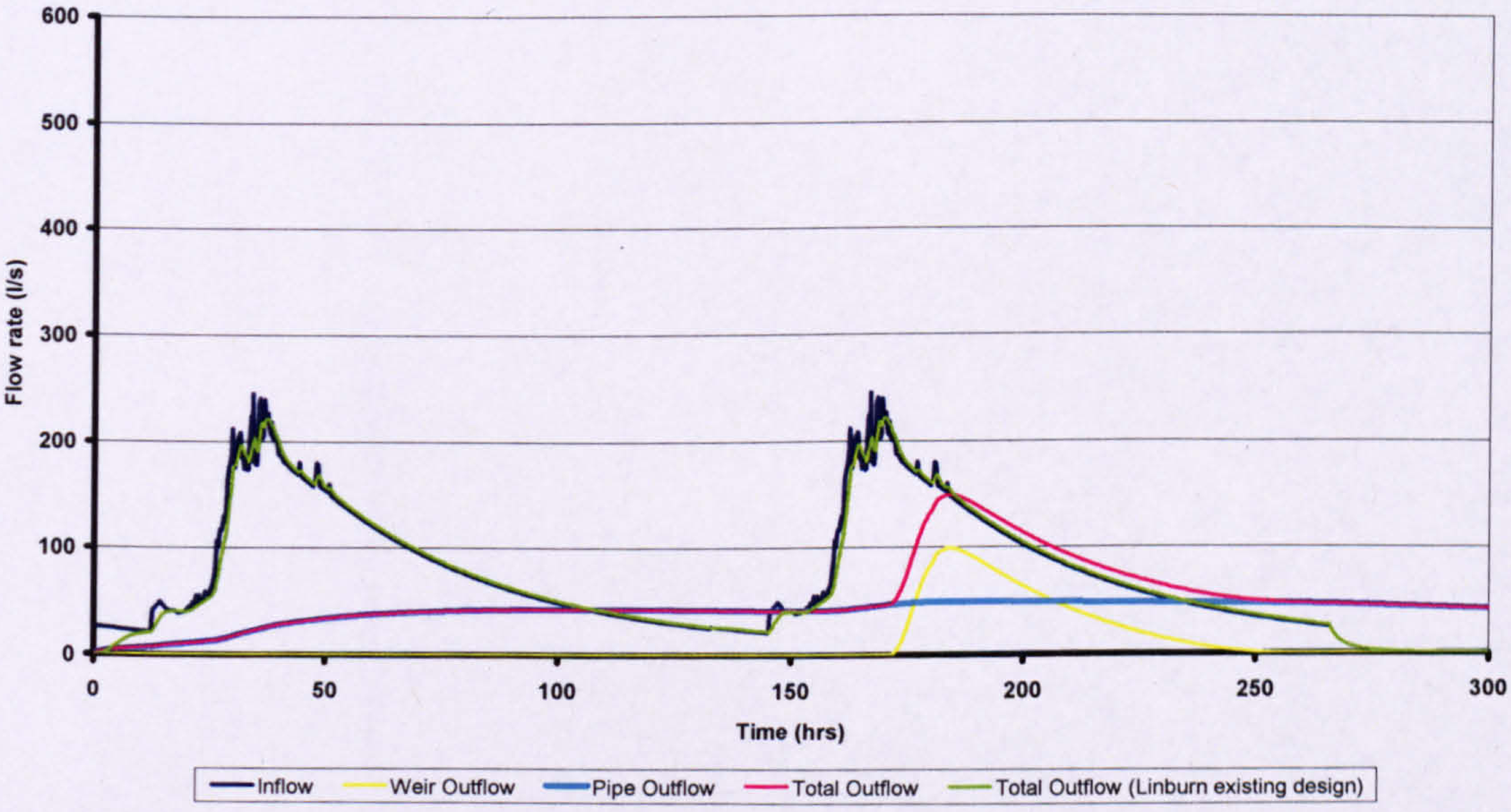


Figure 4.19: Simulations of two consecutive 1 in 2 year events in Linburn Pond under possible climate change scenario (9% magnitude increase).

4.6 Discussion

In the U.K, it is common to design stormwater retention ponds to attenuate single-event design storms (often the 1 in 25 year event is the chosen design storm). However simulations in this chapter have shown that ponds designed in such a manner are likely to fail if consecutive storms occur without adequate drain-down time. The simulations presented here showed that this is dependant upon the amount of storage available before the start of a storm. In the humid, temperate climate of the U.K, it is highly probable that functioning retention ponds will have to operate under conditions of short antecedent periods and short pond draw-down times, particularly in winter. Simulations also showed that the design of the outlet device plays a particularly important role in ensuring that a range of design storms can be attenuated by the pond, i.e. better pond performance for a range of storms was achieved using a dual level outlet device, than using a single level outlet.

Application of the model to Linburn Pond in Scotland showed that it is possible to redesign the pond using multiple event design criteria to greatly improve its flow attenuation performance under climate change. Ideally a designer would also calculate the risk associated with the failure of the pond under multiple event scenarios. However, it is not straightforward to calculate the likelihood of two events of magnitude greater than the 1 in 2 year event occurring with an antecedent period of less than the 6 days, required in this case, to maintain good performance at Linburn Pond. If the two events are independent then the calculation is trivial. However, this is unlikely to be the case for relatively frequent events since they would probably be associated with a single weather system. Further, the above simulations only account for changes in rainfall intensity. Yet it may be that changes in storm intensity due to climate change could be accompanied by a reduction of the antecedent period between events, although no literature could be found to quantify this effect. Clearly, shorter antecedent periods between storm events would negatively impact upon pond performance since this would reduce pond drain down time between events, compromising its ability to provide adequate temporary storage for subsequent storms.

It is thus of crucial importance that the possible climate change scenarios are taken into account at an early stage in the design retention pond systems since, as discussed above, once a pond is in place, it can be difficult to retrofit the system to accommodate the changes. Simulations of Linburn Pond also highlighted the need to thoroughly investigate groundwater levels before implementing pond designs. Groundwater varies spatially and seasonally, however it is generally acceptable that groundwater levels are more likely to be high during the winter months in the North of the UK and lower during the summer months in the South. The example of Linburn Pond has shown how groundwater can adversely affect pond flow attenuation by reducing the volume available for stormwater storage. The inverse of this is also possible in some areas of the UK, where very low ground water levels may encourage pond water to exfiltrate out of the pond to the surrounding groundwater, reducing in-pond water levels, which may adversely affect water quality treatment in a retention pond.

It is possible to increase temporary storage further in a pond such as Linburn with a dual level outlet, either by increasing the pond radius or by increasing the submerged pipe diameter to reduce the drainage time between events. However, it needs to be borne in mind that if the submerged pipe diameter becomes too large, the pond will not attenuate small events. Further, the more rapidly the pond drains to provide good subsequent flow attenuation, the shorter is the residence time of the water in the pond. This is likely to impact on the water quality treatment function of retention ponds, since this is largely governed by the settling times available for sedimentation. Consequently, too large a pipe may result in poor water quality performance. The designer must, therefore, balance the competing functions of a retention pond based on local water resources targets and water quality priorities. These issues are examined in more detail in Chapters 5 and 6.

Finally, it is worth noting that in all of the simulations that have been discussed in this chapter, it has been assumed that the pond volume will remain constant over time. However, as sediment accumulates on the base of a pond, the volume that remains for stormwater storage reduces. The rate at which a stormwater pond will infill depends entirely on sediment production, entrainment and transport rates within the catchment as well as on a number of hydrological, hydraulic and sediment factors such as residence time and particle settling rate respectively However, as simulations earlier in the chapter illustrated, the temporary storage volume in the pond is the single most crucial determinant of flow attenuation performance, which will undoubtedly decrease with increasing pond sediment. This relationship between sediment accumulation and reduction in TSV would enable the failure criterion (used above in describing peak flow attenuation) to be used as an indicator of sediment build up in the pond. Repeated failure - or an increase in failure magnitude while storm inflow conditions lie within the typical range expected for the design storm may be indicative of a reduction in storage capacity beyond the critical value due to sediment accumulation.

The remainder of this thesis considers design of SUDS ponds for combined flow and sediment attenuation. Despite much of the international guidance being based on water Quality governance, there are currently no published design criteria for sediment attenuation in the U.K, and hence no guidelines for pond design for water quality improvement.

5 Modelling Pond Water Quality

This chapter introduces the water quality model developed to simulate sediment retention, and hence, water quality performance of stormwater ponds. The results described in this chapter comprise a sensitivity analysis that was performed using the model to determine the key influences on sediment capture in retention ponds. The findings are presented here and are preceded by a description of the new model.

5.1 Model Development

A water quality model was developed as part of this project to assess the sediment removal efficiency of retention ponds. The model uses output from the flow model (Chapter 3) and calculates the mass of sediment settled for five different particle sizes ranging from 0.001-10mm. The design of the model, described below, was an iterative process and was informed by results of the sensitivity simulations presented in this Chapter. The coding of the model within an Excel spreadsheet was performed by Dr Wallis.

The water quality model simulates the pond by dividing it into two distinct zones (Figure 5.1). Zone 1 is an active flow or transport zone, which extends between the inlet and the outlet. Zone 2 is a quiescent zone, which occupies the areas of the pond where there is no active flow. This conceptual division of the pond recognises that in practice a pond is unlikely to be in a well-mixed condition. Instead the momentum of the inflow, aided by the withdrawal of flow through the outlet, establishes a preferential flow path between inlet and outlet. This phenomenon is frequently referred to as short circuiting. Regions of the pond not occupied by this flow zone provide lateral storage areas. The interface between the zones allows a transfer of suspended sediment to occur between the zones. This two zone representation is similar in principle to the way in which solute transport in rivers is often modelled using a transient storage approach [*Rutherford,* 1994].

The two zones in the pond allow the different conditions and processes that occur under storm conditions in the pond to be modelled. For example, the turbulent flow conditions which are responsible for much of the sediment transport through the pond during the storm are simulated in Zone 1. The quiescent regions of the pond, which are not greatly disturbed by the flow through the pond, are represented by Zone 2. Exchange occurs between the zones and is determined by an exchange coefficient as described in more detail below.

Since particle diameter is a key determinant of sediment settling rate, the water quality model was designed to simulate the fate of up to five different sediment particle sizes. The settling rate is calculated based on the diameter of the particle and the horizontal velocity of water in the zone. The water quality performance of the pond is assessed by determining the mass of sediment retained in the pond during a storm.

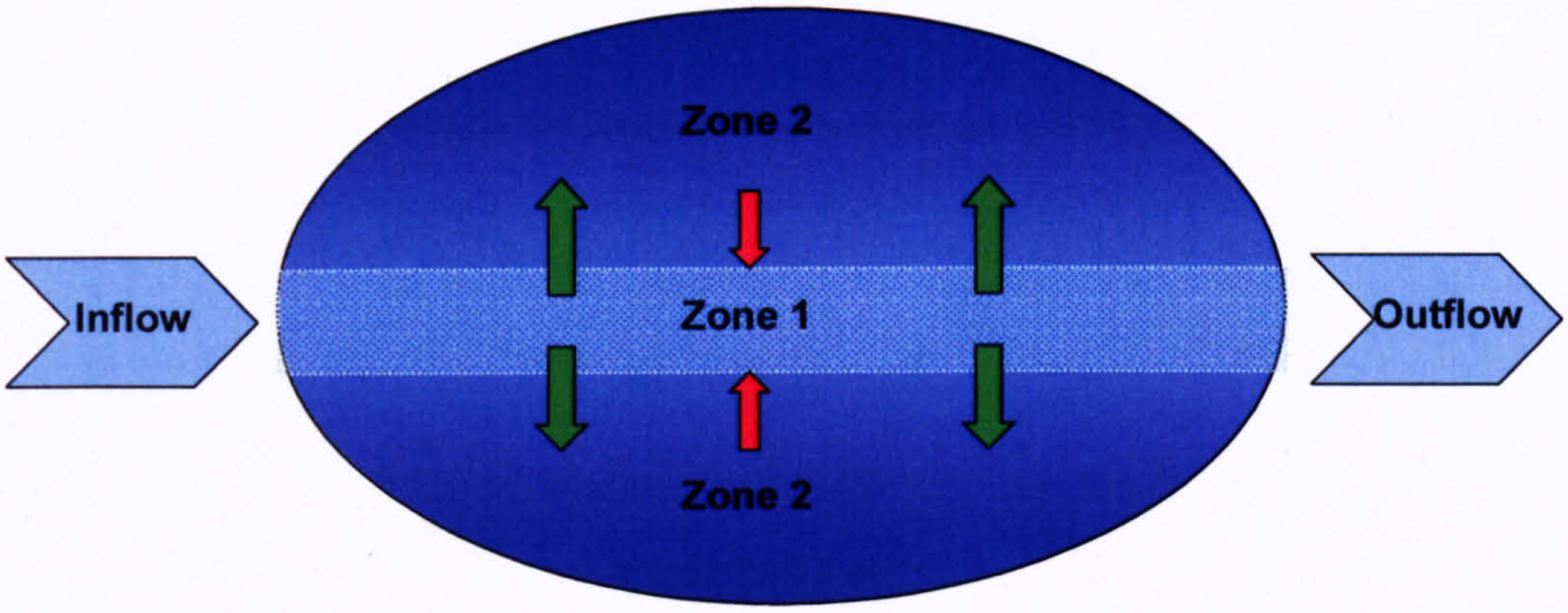


Figure 5.1: Schematic diagram of two zone model showing Zone 1, the transport zone, and zone 2, the quiescent zone. The arrows depict interzone transfer

The mass of sediment retained in Zone 1 is calculated using the conservation of mass described by Equation 5.1 whereby the change of sediment mass in Zone 1 plus the outgoing sediment mass is balanced by the sediment mass entering the pond plus the mass exchanged between the two zones minus the settled mass. Here V_1 is the volume of Zone 1, C_1 is the sediment concentration in zone 1 and C_2 is the sediment concentration in Zone 2. Q_O is the outflow, Qi is the inflow, C_i is the inflow sediment concentration, A_1 is the surface area of Zone 1 and U_1 is the settling velocity of a particle of given size in Zone 1. ε is a constant controlling the sediment transfer rate between the zones, and has units of volume per second.

$$\underbrace{\frac{d}{dt}(V_1C_1)}_{\text{Rate of change of sediment mass}} + \underbrace{Q_oC_1}_{\text{Outgoing sediment mass flow rate}} = \underbrace{Q_iC_i}_{\text{Incoming sediment mass flow rate}} - \underbrace{\varepsilon(C_1 - C_2)}_{\text{Mass flow rate of sediment exchanged between zones}} - \underbrace{A_1U_1C_1}_{\text{Mass flow rate of settled sediment}} \tag{5.1}$$

The mass of sediment retained in Zone 2 is described by equation 5.2 whereby the change of sediment mass in Zone 2 is equal to the mass exchanged between the zones minus the settled mass in this zone. Note there is no incoming sediment in this zone directly from stormwater inflow; nor is sediment carried out of this zone in the outflow. Sediment entering this zone is exchanged from Zone 1 only. Here V_2 is the volume of Zone 2, A_2 is the surface area of Zone 2 and U_2 is the settling velocity of a particle of given size in Zone 2.

$$\underbrace{\frac{d}{dt}(V_2C_2)}_{\text{Rate of change of sediment mass}} = -\underbrace{\varepsilon(C_2 - C_1)}_{\text{Mass flow rate of sediment exchanged between zones}} - \underbrace{A_2U_2C_2}_{\text{Mass flow rate of settled sediment}} \tag{5.2}$$

Each zone is assumed to be well-mixed, but in general the concentrations C_1 and C_2 are different. Equations 5.1 and 5.2 are solved using a similar θ-weighted finite difference scheme to that used for the flow model. Since there are no non-linear terms in the sediment model, no iteration is required but in order to solve for both C_1 and C_2, the equations need to be solved simultaneously. Note that re-suspension of sediment is not included in either zone, since it was assumed to be less important than the other processes. It is also assumed that flocculation or disaggregation of particles does not occur.

The settling velocity of particles in Zone 1 was estimated for a particular particle size from a linear interpolation between (Figure 5.2) the velocity when no settling occurs, V_H, (taken from a modified form of the Hjulstrom curve), and a maximum settling rate, U_Q, that occurs when the water is quiescent. Values of V_H were taken from the line labelled the 'fall velocity' on the Hjulstrom curve (Figure 5.3) which represents the threshold value above which particles remain in suspension due to the turbulent mixing created by the flowing water. The value of quiescent settling, U_Q, is primarily dependent upon on particle diameter and hence weight, as described by Stokes Law [*Chapra*, 1997]. Table 5.1, taken from [*Ellis et al.*, 1995], shows the values used.

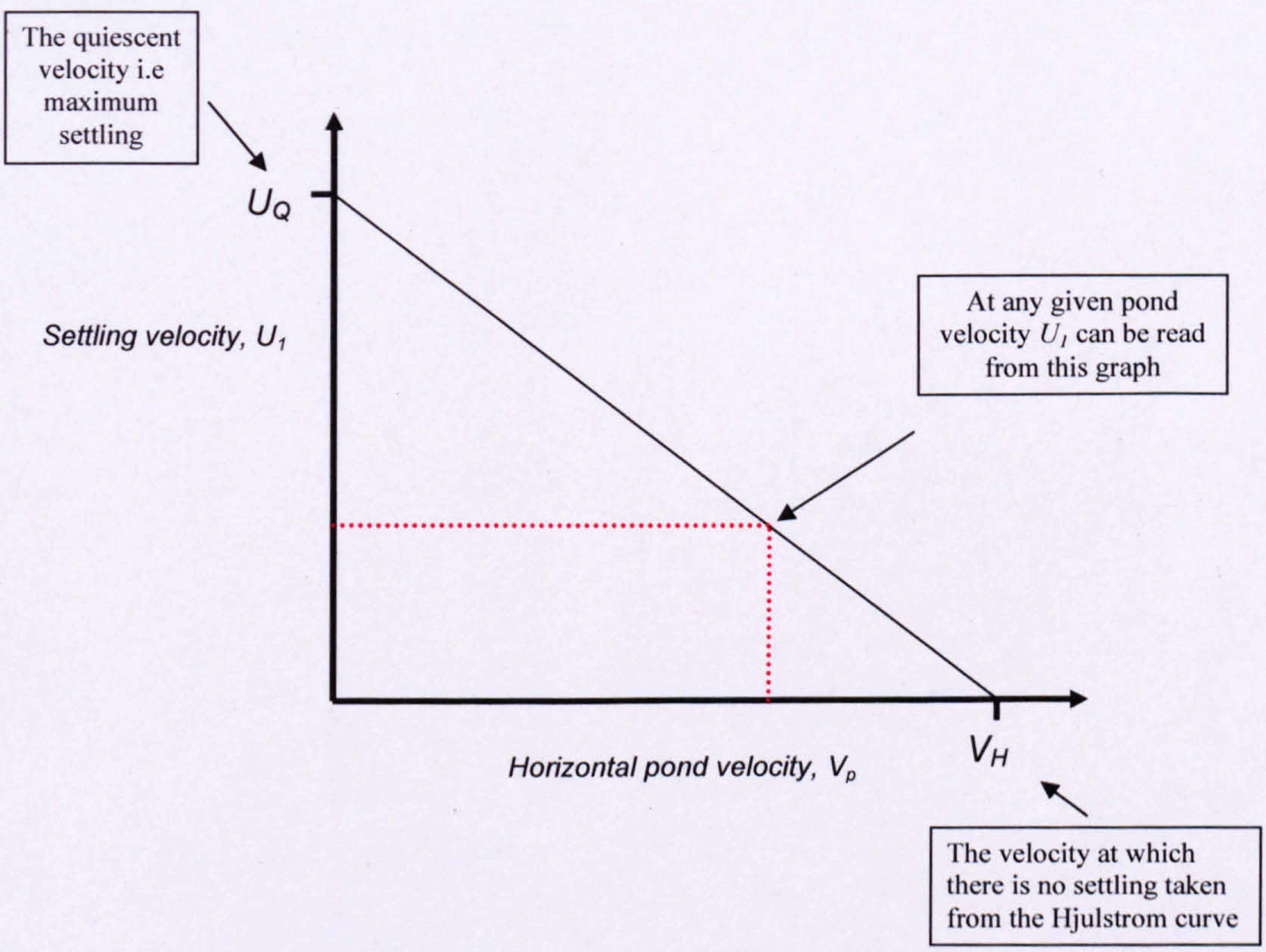


Figure 5.2: Diagrammatic representation of how U_1 is calculated for each particle size using pond velocity, settling velocities from the Hjulstrom curve and quiescent settling velocity.

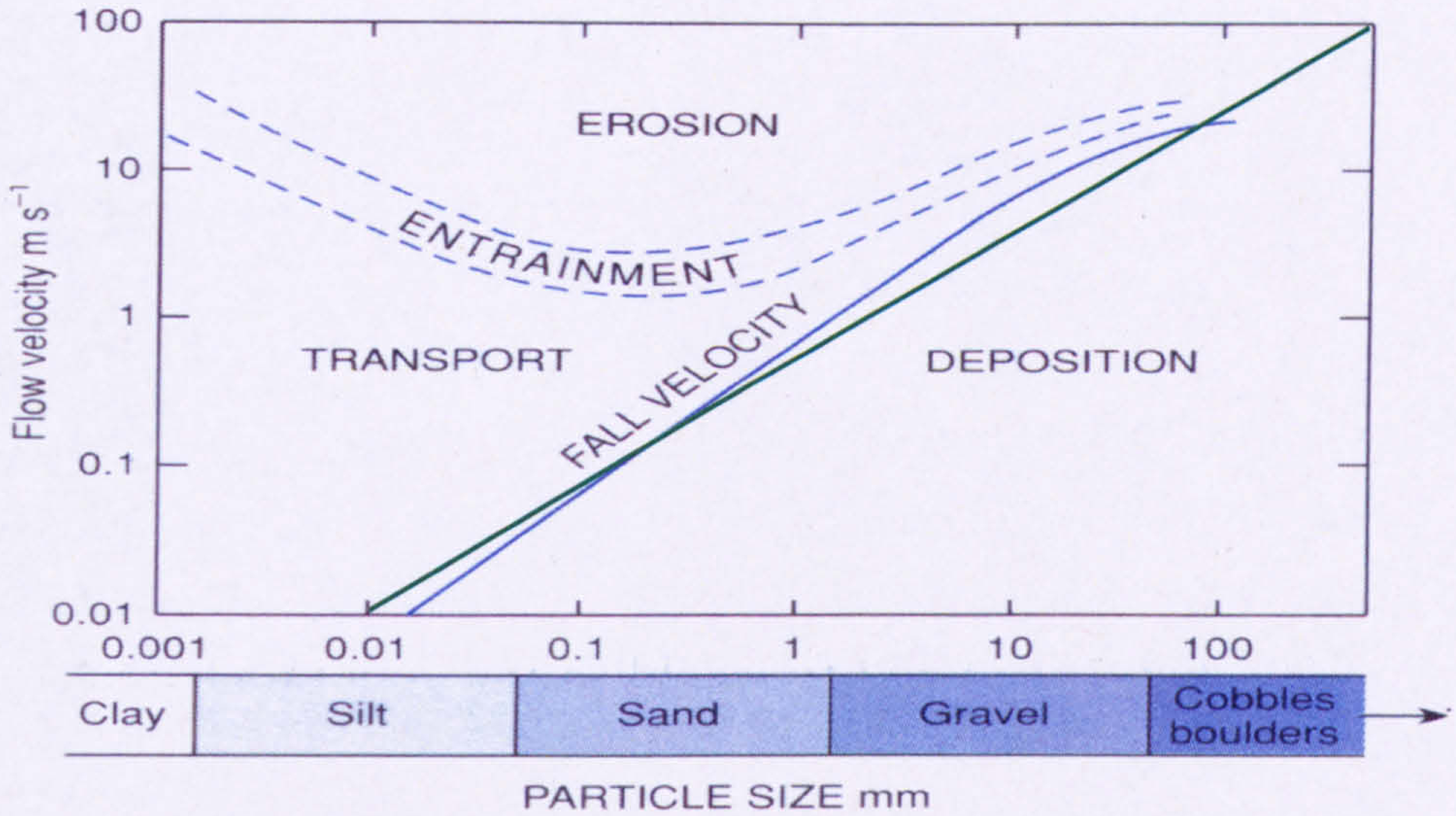


Figure 5.3: The Hjulstrom curve showing the relationship between particle size and the phases of particle motion, modified (green line) for linear interpolation.

Table 5.1: Quiescent settling velocity (U_Q) for particle diameter in the range 0.001-10mm [*Ellis et al.*, 1995].

Diameter (mm)	Settling Velocity (m/s)
10	0.8
1.0	0.2
0.1	0.01
0.01	0.0002
0.001	0.00001

With reference to Figure 5.2, the horizontal velocity of the water in the pond (V_P) was calculated by dividing the distance travelled by the water by the time taken for the water to flow through the zone (Equation 5.3). The former is given by the length of Zone 1 (the diameter of the pond = 2R, where R is the pond radius) and the latter is given by the retention time of the water in Zone 1 (V_1/Q_o). Although the diameter of the pond is a constant, the retention time changes during a storm event. Hence V_p changes in a manner that reflects reality i.e. it is zero when there is no outflow and it reaches a maximum under high outflow conditions. Consequently, the settling velocity in Zone 1 approaches U_Q during periods of low flow, but is reduced during periods of high flow.

$$V_P = \frac{2Q_o R}{V_1} \qquad (5.3)$$

Note that in Zone 2, settling always occurs under quiescent conditions at the velocity shown in Table 5.1.

The particle size distribution, the volume ratio of the two zones and the inter-zone transfer rate (ε) were obtained using data from [*Hepburn*, 2004]. The study used various sources to characterise the average particle size distribution found in the Linburn and Halbeath Pond catchments and to identify the source of the sediments. The average particle size distribution at Linburn Pond was characterised as that arising from residential and/or commercial areas based on data from [*Butler et al.*, 1993]. Furthermore using data from a field monitoring program at the pond by [*Spitzer and Jeffries*, 2003], the study by [*Hepburn*, 2004] illustrated that typically 65% of the sediment that enters Linburn pond passes straight out of the outlet (i.e. only 35% is captured). This data was used to calibrate the sediment capture efficiency of the model, by adjusting the values of the volume ratio and ε, so that when applied to Linburn Pond, the model reproduced the 35% capture level. To obtain this value, the volume ratio used was 0.15 and the value of ε required was 0.01. Table 5.2 shows the particle size distribution used in all water quality simulations.

Table 5.2: Particle size distribution used in water quality simulations

Particle diameter (mm)	Fraction (%)
0.001	79
0.01	10
0.1	0.1
1	0.9
10	0.7

5.2 Sediment capture in retention ponds: a sensitivity analysis

A sensitivity analysis was conducted to determine the primary influences on sediment attenuation in retention ponds. Following the method used in Chapter 3 for assessing the dominant influences on flow attenuation, appropriate performance indicators were selected to determine the effect of varying several pond parameters on the percentage of sediment mass that entered the pond, which settled.

5.2.1 Introduction

In the simulations described below a generic retention pond is used to investigate the sensitivity of sediment retention to certain basic design parameters. The simulations were undertaken using the two-zone sediment model described above with flows being provided by the flow model described in Chapter 3. Having designed a generic base case, the effect of changing the following five parameters was considered:

- pond radius
- pipe elevation
- outflow pipe diameter
- initial water level
- sediment inflow distribution (whilst keeping total incoming sediment mass constant)

These simulations not only provided information on pond behaviour, but also provided confirmation that the physical representation of sediment deposition in the model was sound and that the model contained no significant errors of either a computational or logical nature.

The base case pond had a radius of 30m, a 90° v-notch weir situated 3m above the base of the pond, and an outlet pipe of diameter 0.1m at an elevation of 1.5m above the base of the pond. The base case was specifically designed with a dual outlet, incorporating the optimum design features presented in Chapter 4. The base case inflow hydrograph had a peak inflow of 125 l/s (equivalent to the 1 in 2 year storm event introduced in Chapter 4) and a duration of 24 hours, which represents a storm capable of mobilising all sediment particles sizes that have accumulated on the urban surfaces in the pond's catchment. The base case inflow sedigraph (concentration-time distribution) was triangular with a peak concentration of 100mg/l and a duration of 9.6 hours. With this, all of the sediment enters the pond in the first 40% of the inflow period (24 hours), and represents a first-flush type of scenario. The total mass of sediment entering the pond in all simulations in this chapter was 86400g.

All simulations used a time step of 0.24 hours. This was deemed to be a suitable time step value based on an assessment of the sensitivity of the base case simulations to changes in time step. All simulations were run until all the suspended sediment in the pond had either been flushed out or had settled (after the cessation of the outflow).

In addition to the base case itself, 24 simulations were undertaken, each of which was identical to the base case except that one of the five controlling parameters introduced earlier was varied. The results of the sensitivity analysis are presented in Section 5.3.

5.2.2 Performance Indicators

Indicators of flow attenuation performance have been discussed in detail in Chapter 4, however since both flow and water quality parameters are discussed in the following results sections, a brief review is provided here. The main indicator of flow attenuation performance is the Peak Flow Ratio. This is the ratio of the peak outflow to the peak inflow. It was used in Chapter 4 to determine whether the flow attenuation failure criterion had been met by a pond for a particular storm. Good flow performance is achieved when the Peak Flow Ratio is below 0.5. Any ratio greater than this, is indicative that the peak flow has not been reduced by at least 50%.

Since sediment is a key vector for pollutant transport in stormwater runoff, water quality performance was assessed primarily as the percentage of the total mass of sediment in the inflow that settled out of suspension, referred to as the Total Mass Settled. In the results tables that follow the percentage mass settled in each pond zone is shown as well as the percentage total mass settled.

5.2.3 Processes and modelling

It is generally accepted that the two most important physical parameters that influence the settling of suspended sediment are the hydraulic retention time [*Chu et al.*, 2005; *Holland et al.*, 2004; *Kadlec and Knight*, 1996; *Persson*, 2000; *Walker*, 1998] and the settling velocity. The former is a description of how long the water remains in the pond and hence indicates the length of time available for settling. It is inherently linked to the storage and outflow characteristics of a pond. The settling velocity describes how quickly sediment falls out of suspension. It is primarily dependent on particle size and the nature of the flow, with settling occurring more quickly under no flow (quiescent conditions) than when flow occurs (dynamic conditions).

The way in which retention time and settling velocity control sediment settling is relatively simple under steady flow conditions. During steady flow the inflow and outflow rates of water are constant and equal to each other. The volume of water in the pond and the retention time (evaluated as the volume/flow rate) are also constant as is the settling rate (for any given particle size). These sort of conditions prevail in wastewater treatment settling tanks. In stormwater retention ponds, however, the inflows and outflows are unsteady and unequal; and the volume of water, the retention time and the settling rate are all temporally variable (settling is reduced under high flows when compared to low flows), as detailed in Section 5.1.

Despite these complexities, the amount of sediment captured by a pond is determined by two simple mechanisms: flushing and settling. Flushing is the process by which sediment is transported through and out of the pond in the outflow, and is quantified by the product of outflow and sediment concentration. Any sediment lost in this way cannot be captured by the pond. In contrast, settling is the process by which sediment capture occurs and is quantified by the product of pond surface area, settling velocity and sediment concentration. Clearly, the amount of sediment that settles out in the model can be expected to depend on: the amount that is brought into the pond in the inflow; the (variable) outflows; the (variable) hydraulic and settling conditions in the pond; and the length of time between inflow events. The latter is important because it is unlikely that all the sediment will settle before the outflow ceases, i.e. quiescent settling continues in the permanent pool until either all the sediment has settled or the next inflow event occurs. Note that once the outflow has reduced to a small enough value, quiescent settling occurs in both zones and eventually suspended sediment concentration in both zones becomes equal. Finally, it is assumed that no re-suspension of settled sediment occurs.

5.2.4 Retention time

Although retention time was not explicitly used in the sediment model, its influence on the settling is implicit and should be identifiable from the model results. However, it is not clear how a representative retention time is best evaluated for a retention pond subject to an inflow event. The importance of retention time has been highlighted by [*Chu et al.*, 2005; *Walker*, 1998] but a universal method for evaluating retention time under these conditions has not been developed.

In this thesis the concept of minimum retention time has been developed, based on conditions in Zone 1. This provides an indication of minimum retention time in the pond since Zone 1 is the active transport zone between the inlet and the outlet. Figure 5.4 shows inflow, outflow, pond volume and retention time results from the base case described in Section 5.2.1, with the retention time being evaluated as the ratio of pond volume to outflow at every time step of the simulation. The retention time varies over time with a rapid decrease from an initially very high value, followed by a relatively constant period and then a rapid increase towards the end of the storm event. The large retention times are clearly associated with the very low outflows that exist during the early part of the inflow and during the last few hours of the outflow (before it ceases, due to the water level returning to its original value of 1.5m).

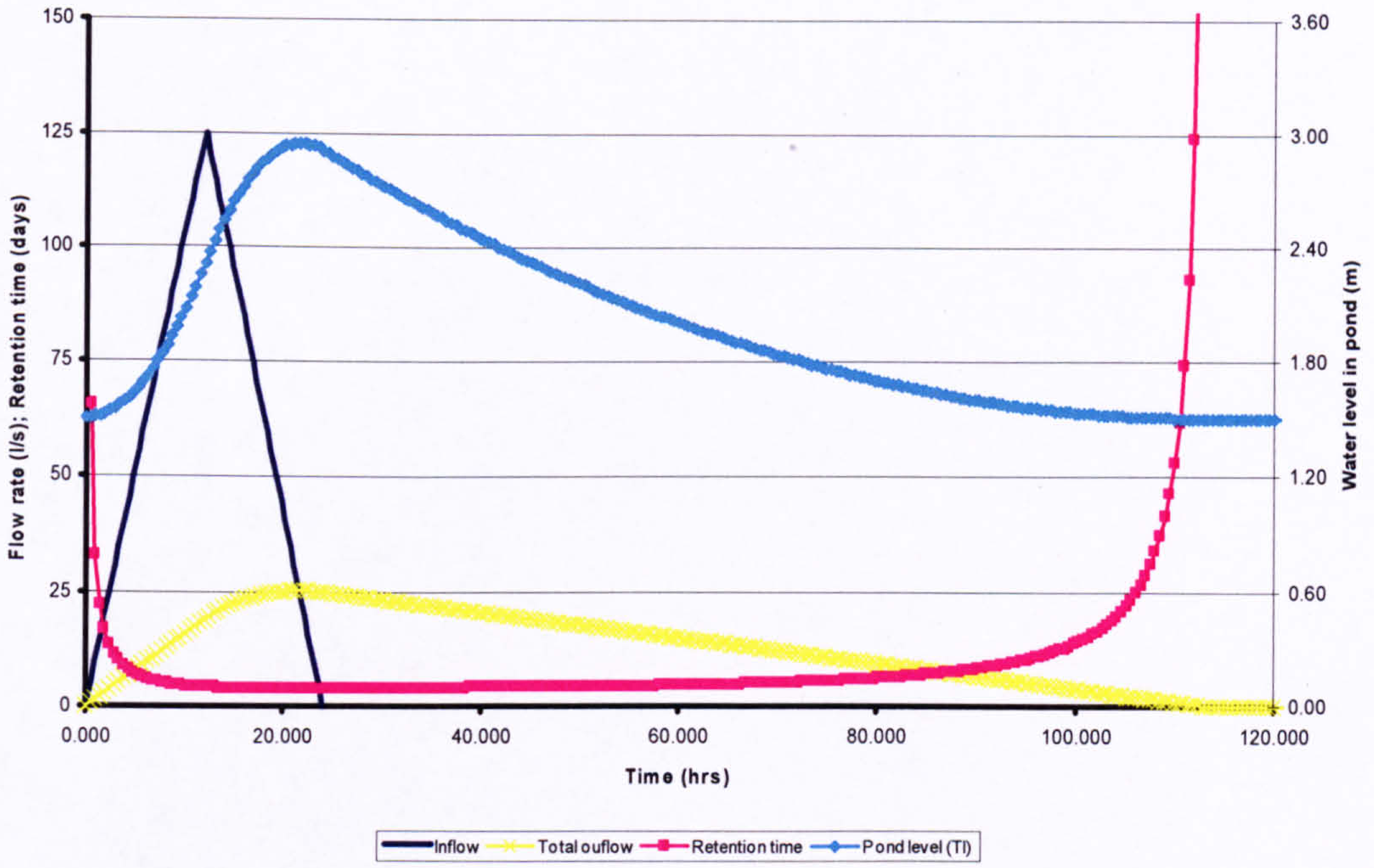


Figure 5.4:Typical variation of hydraulic conditions in a retention pond: model results for base case.

In all the simulations conducted the minimum retention time corresponded to the time of peak outflow (and maximum storage), and for the majority of the time the retention time was not significantly larger than its minimum value. For example, for the data in Figure 5.4, the retention time was: less than 10% larger than the minimum between about 10 and 54 hours; less than 50% larger than the minimum between about 6 and 78 hours; and less than 100% larger (i.e. double) than the minimum between about 4 and 88 hours. Although using the minimum retention time might not be the optimum way of characterising the retention time of each simulation, it is not clear that there is a better one, and it also has the advantage of simplicity.

The suitability of using the minimum retention time to characterise sediment settling is explored in Figure 5.5, which shows the percentage mass settled plotted against the minimum retention time for all the sensitivity simulations undertaken (and described in the following sections). Although there is scatter in the data, there is a positive association between the minimum retention time and the mass of sediment settled. Tables describing the sensitivity analysis results for retention time in full can be found in Appendix E.

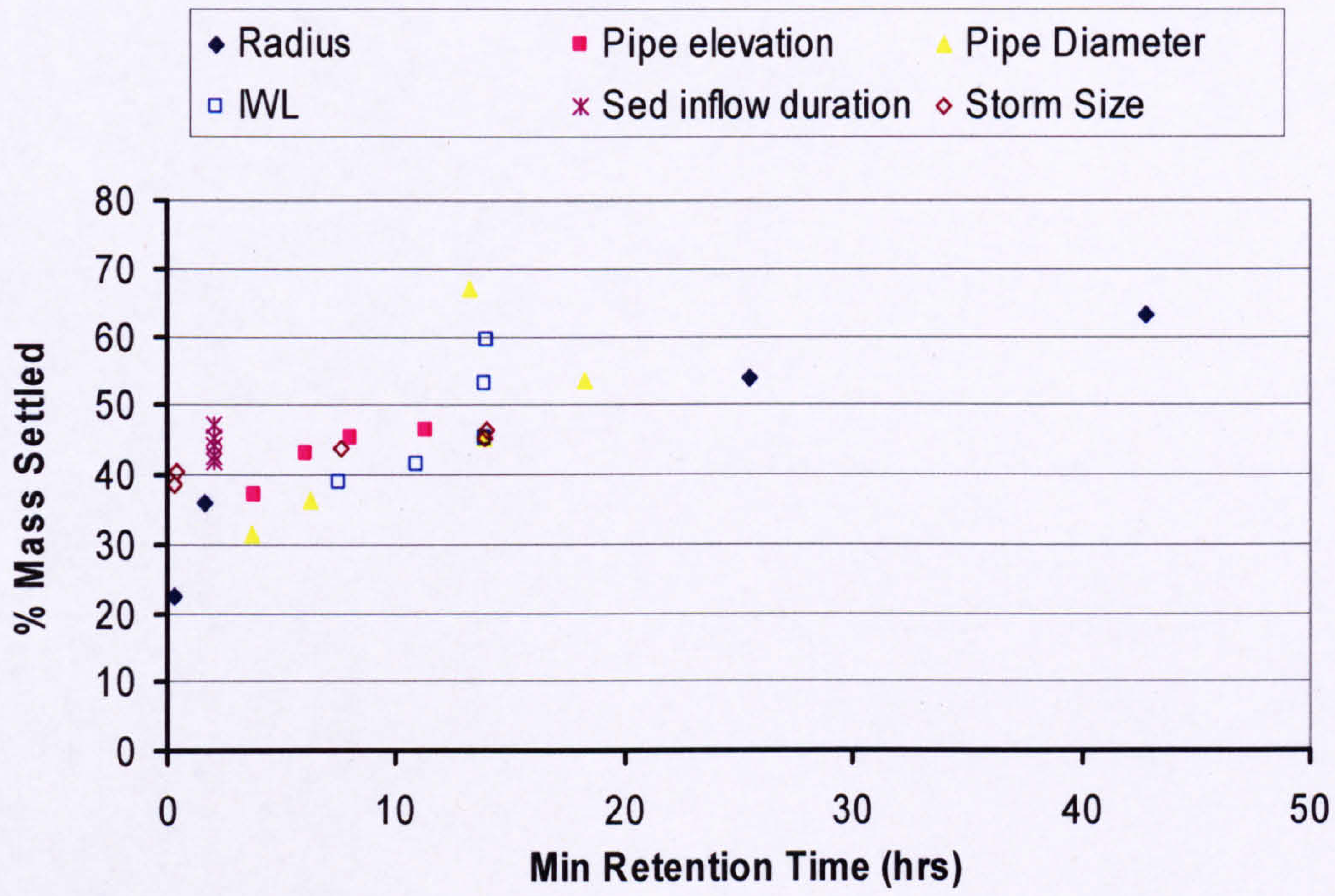


Figure 5.5: Correlation between sediment mass settled and minimum retention time for all sensitivity simulations in which different parameters were varied.

5.3 Sensitivity Analysis

The results described in this section focus on the behaviour of sediment, but the operation of the dual outlet is also discussed because this has an important influence on the sediment capture performance of a pond. An essential feature is that sediment flushed out of a pond in the outflow cannot be captured, and therefore periods of high outflow tend to reduce sediment capture. Such occasions are frequently associated with the operation of the weir. Once outflow ceases (i.e. when the pond has drained down to the pipe outlet), any sediment remaining in suspension will eventually settle, and hence will be captured.

Figures 5.6 – 5.9 show an illustrative set of results from the sediment model, which demonstrate some points common to many of the simulations. The results shown are for a 50m pond radius with the other parameters being as for the base case described in section 5.2.1. A large radius case is used here to avoid the added complexity of the operation of the weir. The flow results shown in Figure 5.6 show a very good flow attenuation provided by the pipe outlet. Similarly, Figure 5.7 shows a significant reduction in outflow sediment concentration and a spreading out over time, compared to the sediment inflow.

Figure 5.8 shows the mass of the smallest sediment size fraction (0.001 mm) settling in both zones in the pond over time. For this particle size, a similar total mass of sediment settles in both zones (indicated by the areas under the curves), but the distribution over time is different, with the peak settling mass transport rates having different magnitudes and occurring at different times. Finally, the concentration of each particle size in Zone 1 is shown in Figure 5.9 (as stated earlier, these are equal to the concentrations in the outflow). There is a clear difference in settling behaviour between the two smallest particle size fractions, 0.001mm and 0.01mm and the larger fractions (0.1-10mm) with a greater concentration of the two smallest particle sizes, in the outflow of the pond. Such findings are not unexpected since these two particle sizes remain in suspension even under conditions of very low velocity flows, as shown by the Hjuslstrom curve (Figure 5.3). While large particle sizes settle out almost immediately, the smallest two fractions remain in suspension for longer periods of time and are available for transport out of the pond, in the outflow. In the following sections the discussion focuses on the total sediment captured rather than on any differences between the sediment captured in the two zones.

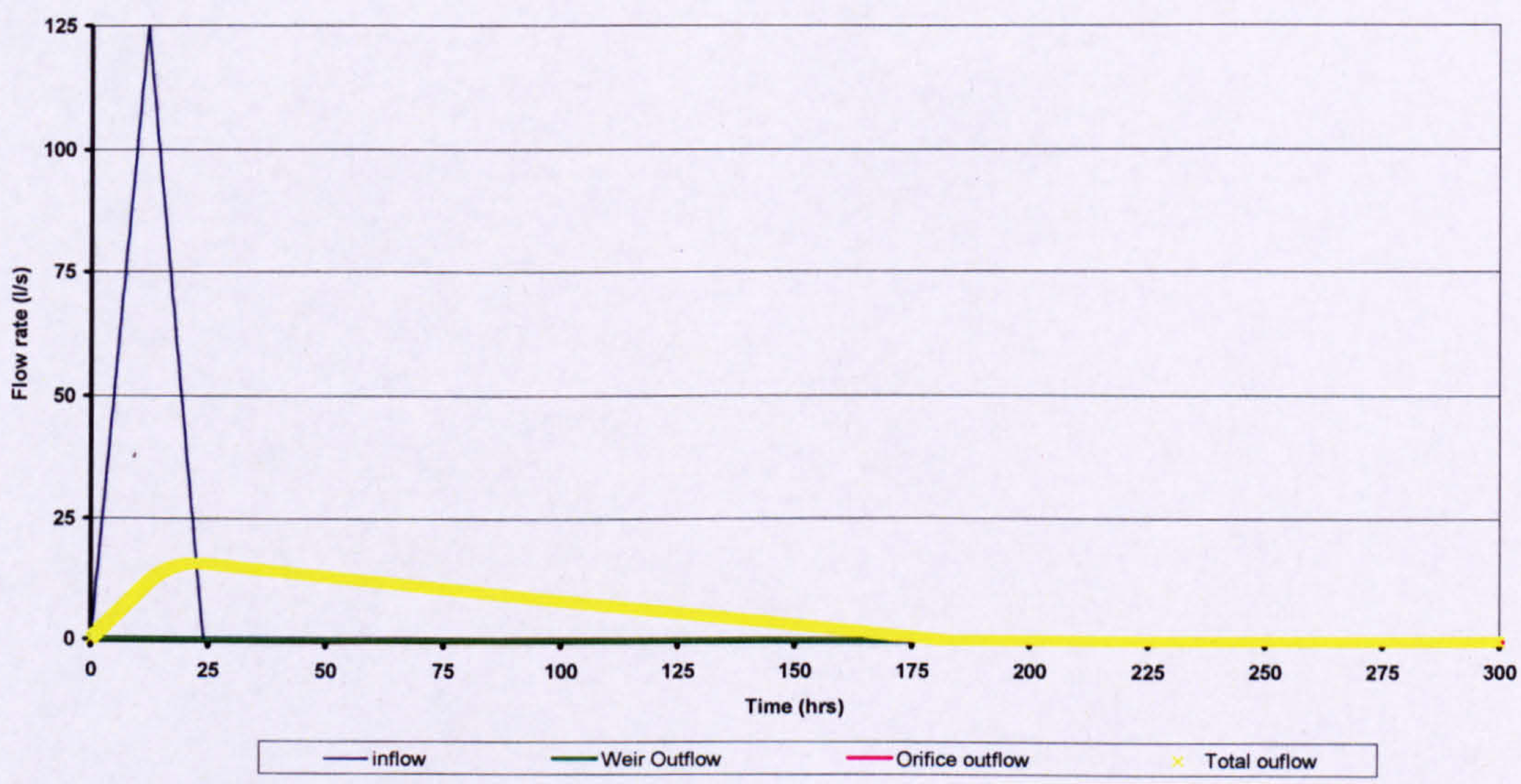


Figure 5.6: Inflow and Outflow for a 50m radius pond.

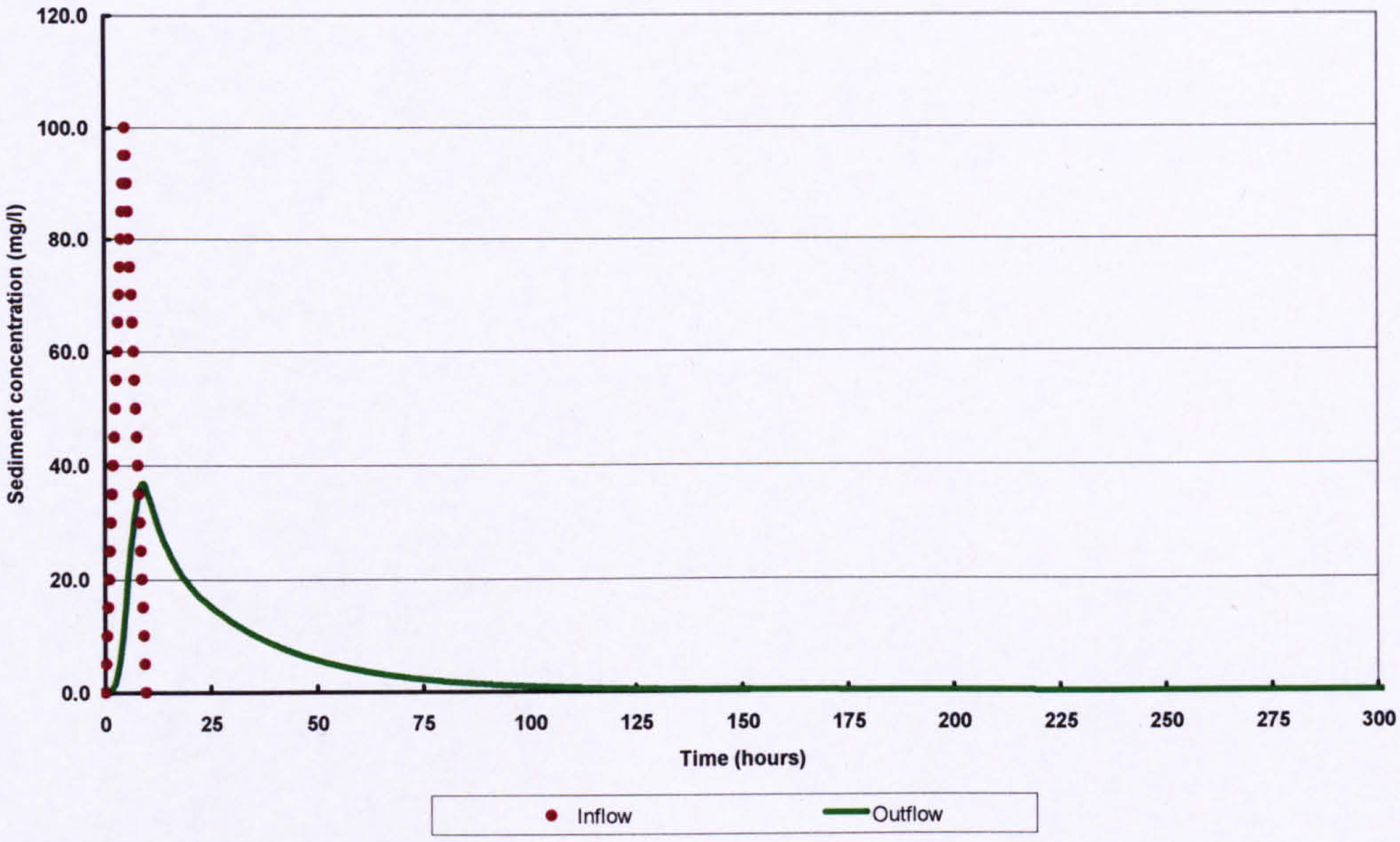


Figure 5.7: Sediment inflow and sediment outflow concentrations in a 50m radius pond.

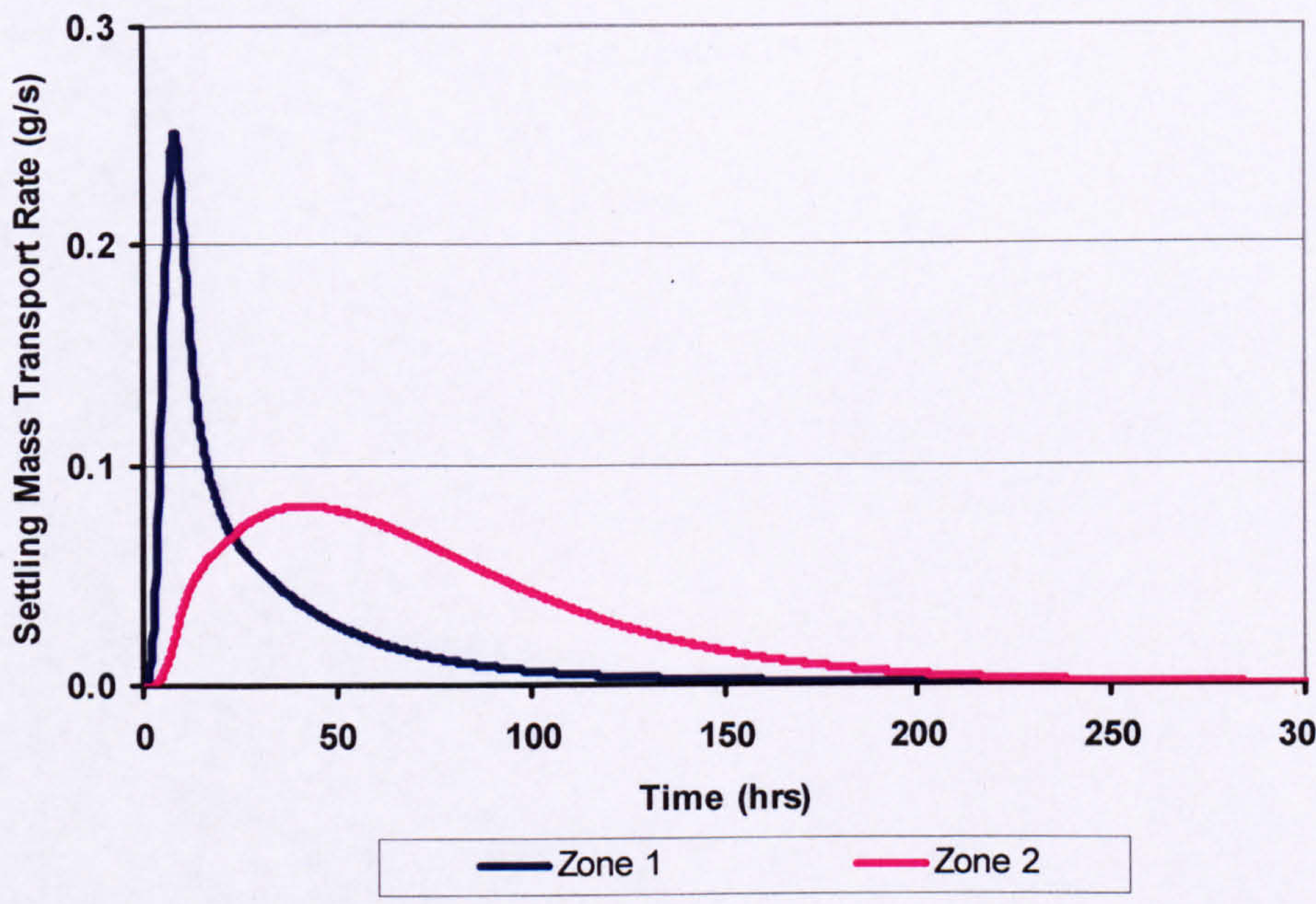


Figure 5.8: Mass of 0.001mm sediment fraction settling in zones 1 and 2 in a 50m radius pond.

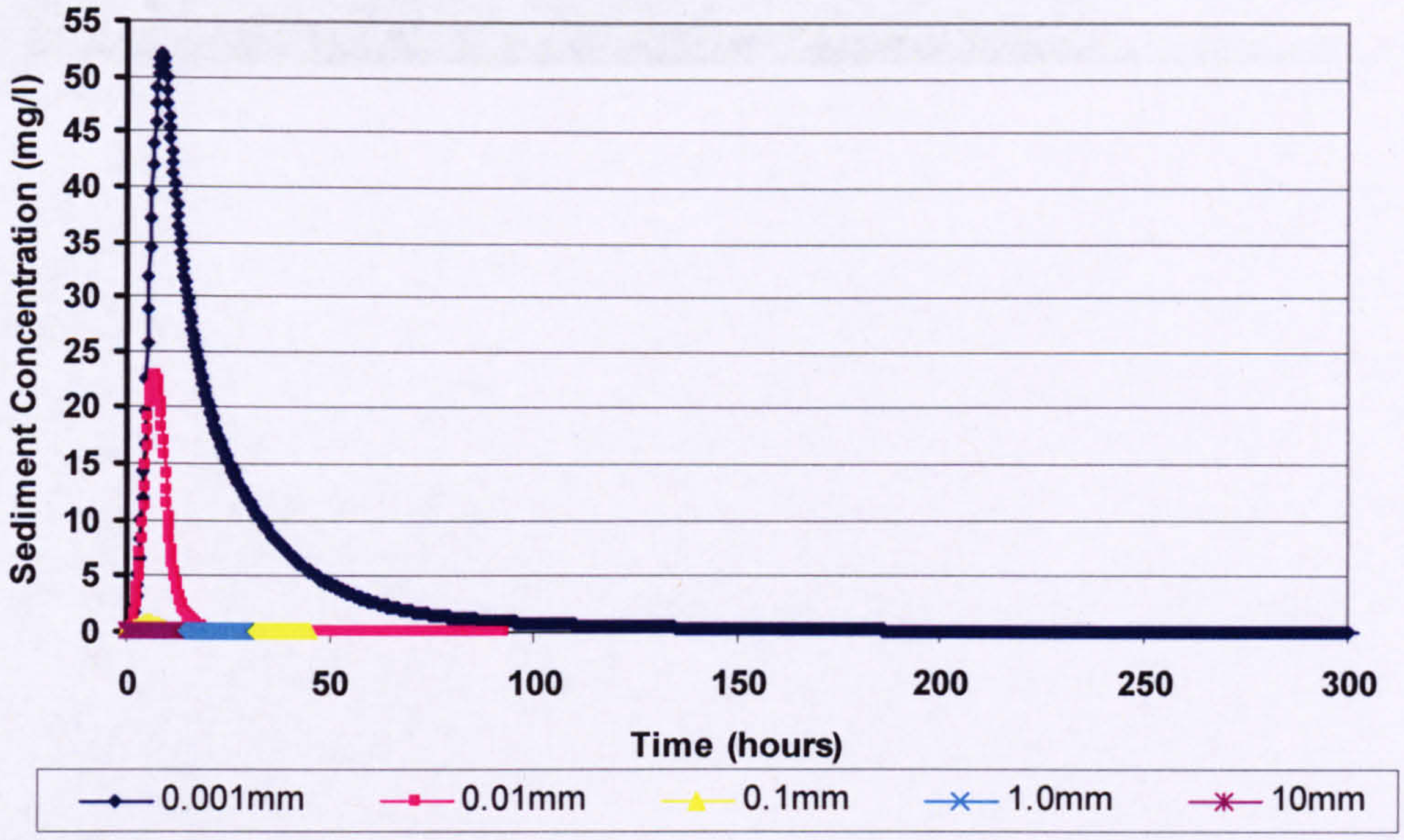


Figure 5.9: Concentration of each particle size in the outflow of a 50m radius pond (zone 1).

5.3.1 Varying Pond Radius

From Chapter 4, it was clear that pond radius has a large influence on the flow attenuation of retention ponds, since it exercises a key control on the Temporary Storage Volume (TSV) – the storage volume between the permanent pool and the weir crest available before the start of a storm. However it is unclear how much influence pond radius has on the water quality performance of ponds. In this analysis pond radius was varied in order to determine its effect on water quality performance. The radii considered ranged from 10m - 50m. Other parameters were held fixed at the base case values (section 5.2.1).

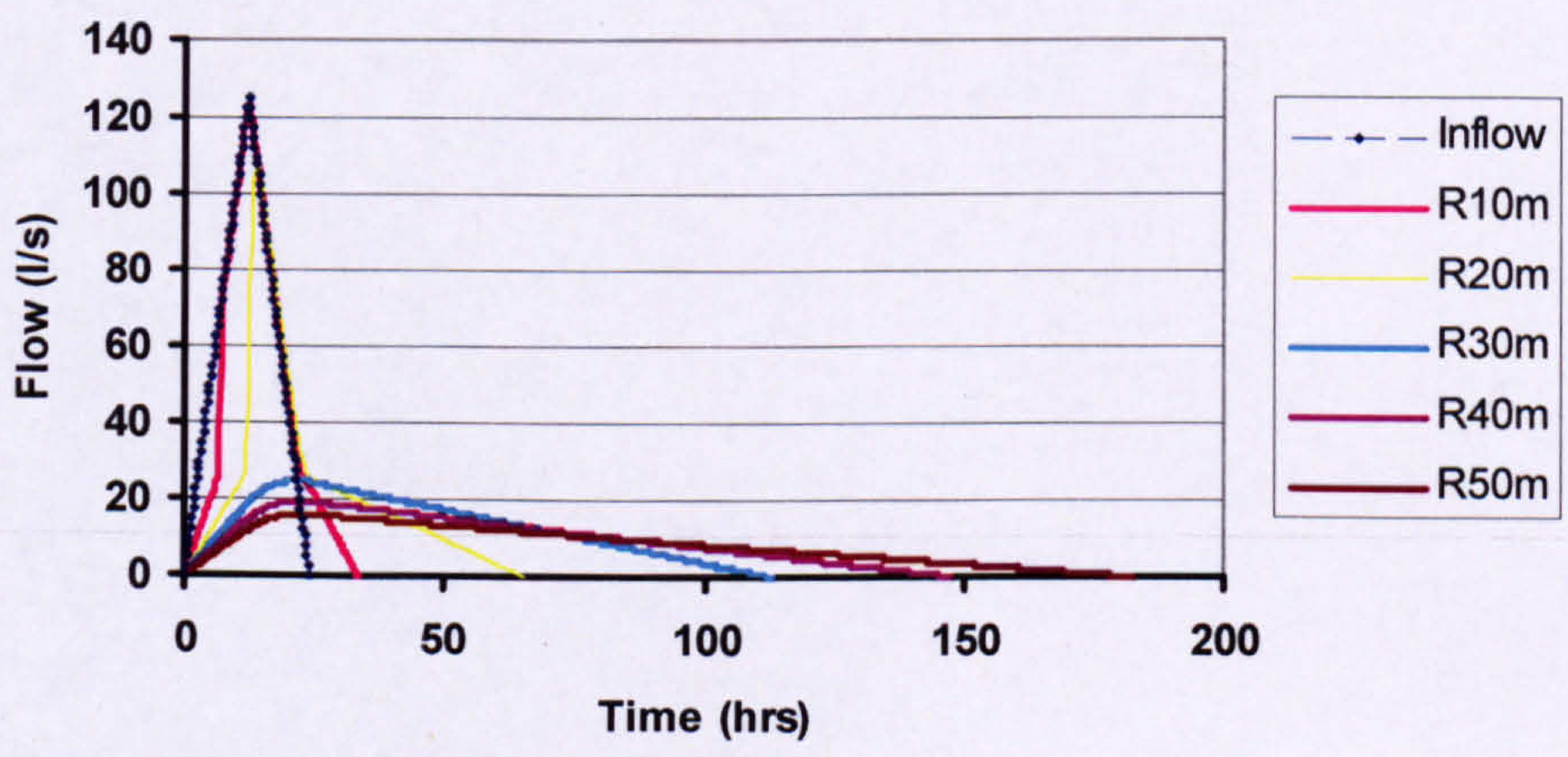


Figure 5.10: Inflow and Total Outflow for 5 pond radii (ranging from 10-50m) .

Table 5.3: Flow and water quality results from 5 ponds with different radii.

Radius (m)	10	20	30	40	50
Flow					
Peak Outflow (l/s)	123.75	106.95	25.09	19.47	15.90
Peak Flow Ratio	0.99	0.86	0.20	0.16	0.13
Min Retention Time (hrs)	0.35	1.62	13.83	25.50	42.82
Mechanism of Outflow	Weir and Pipe	Weir and Pipe	pipe only	pipe only	pipe only
Water Quality					
Total mass settled (%)	22.50	22.90	32.20	53.70	63.32
Total mass in Inflow (g)	86400	86400	86400	86400	86400
Total Mass in Outflow (g)	66987	66572	58552	39967	31740

As pond radius increases, peak outflow decreases while outflow duration increases, see Figure 5.10 and Table 5.3. This occurs because the fixed inflow volume is being distributed over a larger surface area and therefore the depth of water in the pond (and hence the head over the outflow devices) is reduced. Such a reduction in head, results in a decrease in peak outflow with increasing pond radius. It can also be clearly seen from Figure 5.10 that the smaller pond radii cases (10 and 20m) have much higher peak outflows than the other ponds. This is because the ponds are too small (with respect to the inflow volume) to contain and drain the event using only the pipe and thus the weir operates in these two cases. When the weir is in operation, outflows are larger than when the drainage occurs through the pipe alone. For the three larger pond radii, the ponds are large enough to attenuate the storm and drain it completely through the pipe alone, and thus the water level in the pond never reaches the crest of the weir.

Table 5.3 summarises the results from the sediment model. It can be clearly seen that the percentage total mass of sediment settled in the pond increases for larger pond radii. This occurs due to two key effects: a reduction in sediment concentration in the pond as the permanent pool volume increases; and an increase in TSV, which leads to the weir not being used.

The first effect – a reduction in pond sediment concentration - occurs due to increases in the surface area of the pond as the radius increases. Since the pipe outlet is fixed at an elevation of 1.5m, the permanent pool volume likewise increases. This results in more dilution of the incoming sediment mass, and hence leads to reducing sediment concentrations in the outflow of larger radius ponds.

The second effect – an increase in TSV – controls the pond outflow. Cases where the weir operates (radii of 10m and 20m) are those where TSV is inadequate to store the storm volume. This produces much larger outflows in these two cases than in ponds with larger radii (Table 5.3). In general, outflows decrease with increasing radius due to reducing heads over one or both outlet devices. As the pond radius increases greater dilution and smaller outflows both contribute to a reduced sediment mass being lost through the outlet(s). Hence a larger percentage of the sediment remains in the pond and eventually settles. This trend is illustrated well in Figure 5.11.

Also shown on Figure 5.11 is the minimum retention time in each pond. It is apparent that retention time increases with increasing pond radius. In particular, the retention times for the two smallest ponds of 2.4 and 10.8 hours, respectively, where the weir operates are considerably smaller than the retention times of 92.2-285 hours for the three largest ponds. Under conditions of low retention time, much of the contaminant load is flushed straight through the pond in the outflow. For example, in the 10m radius pond only 23% of the sediment mass settled in the pond compared to 63% for the 50m radius pond.

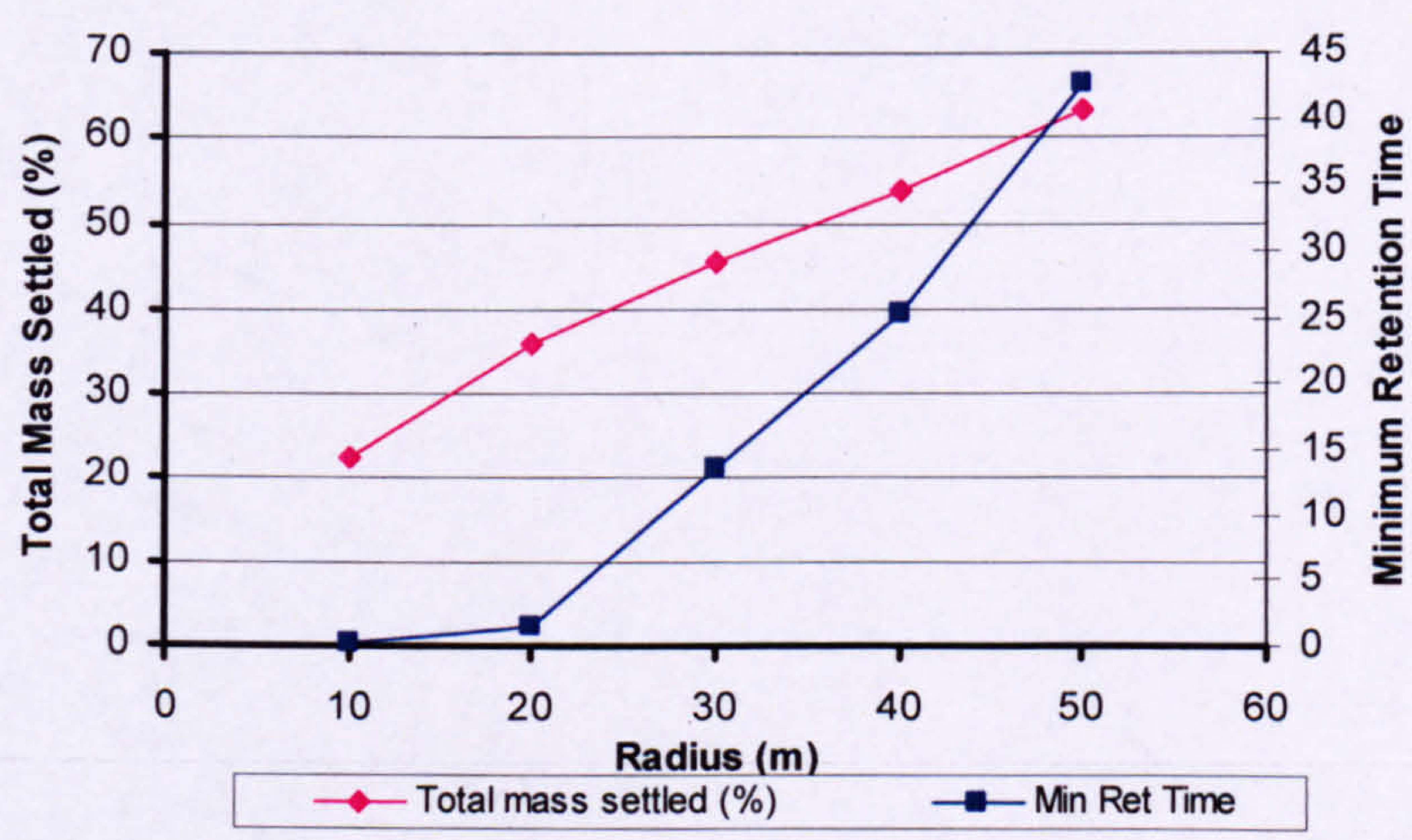


Figure 5.11: Percentage of the total Mass of Sediment Settled and Minimum Retention Time with increasing pond radius.

5.3.2 Varying Pipe elevation

The pipe elevation is critical in determining two things in the retention pond. Firstly, it determines the minimum level that the pond can drain down to (and thus the permanent pool level) and secondly it determines the TSV, the storage volume remaining between the permanent pool and the weir crest. In this analysis the effect on water quality performance of varying the elevation of the pipe was investigated. In each case it was assumed that the water level had adequate time since any previous storm to drain down to the pipe elevation before the start of the inflow event, and thus the initial water level was always at the elevation of the pipe at the start of the simulation. The pipe elevation was varied from 0.5m to 2.5m above the base of the pond. All other pond parameters were held fixed at the base case values.

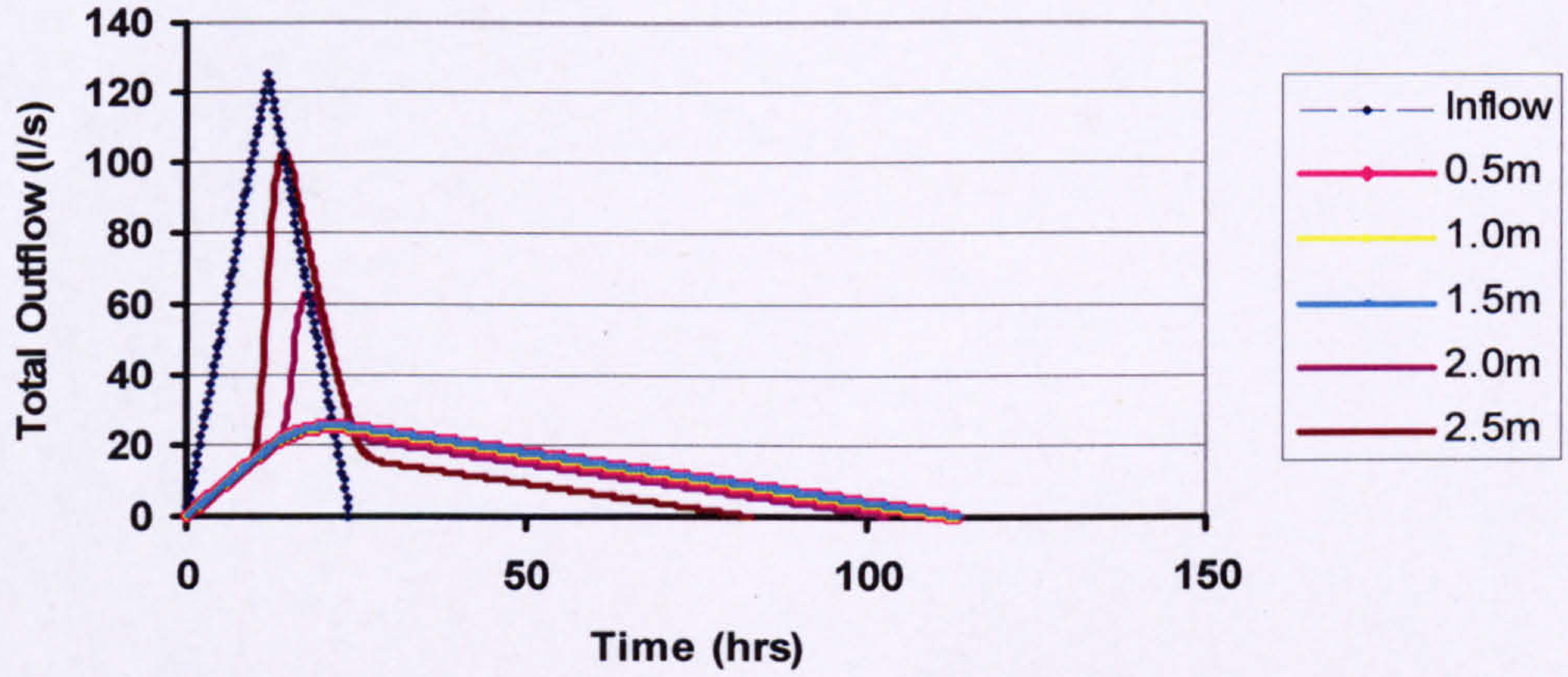


Figure 5.12: Inflow and Total Outflow for 5 pipe elevations.

Table 5.4: Flow and water quality results from ponds with different pipe elevations.

Pipe Elevation (m)	0.5	1	1.5	2	2.5
Flow					
Peak Outflow (l/s)	25.09	25.09	25.09	63.02	102.23
Peak Flow Ratio	0.20	0.20	0.20	0.50	0.82
Min ret Time (hrs)	7.98	11.29	13.83	6.06	3.83
Mechanism of outflow	pipe	pipe	pipe	pipe & weir	pipe & weir
Water Quality					
Total mass settled (%)	46.46	45.26	45.25	42.84	37.03
Total mass in Inflow (g)	86400	86400	86400	86400	86400
Total Mass in Outflow (g)	46276	47293	47305	49429	54406

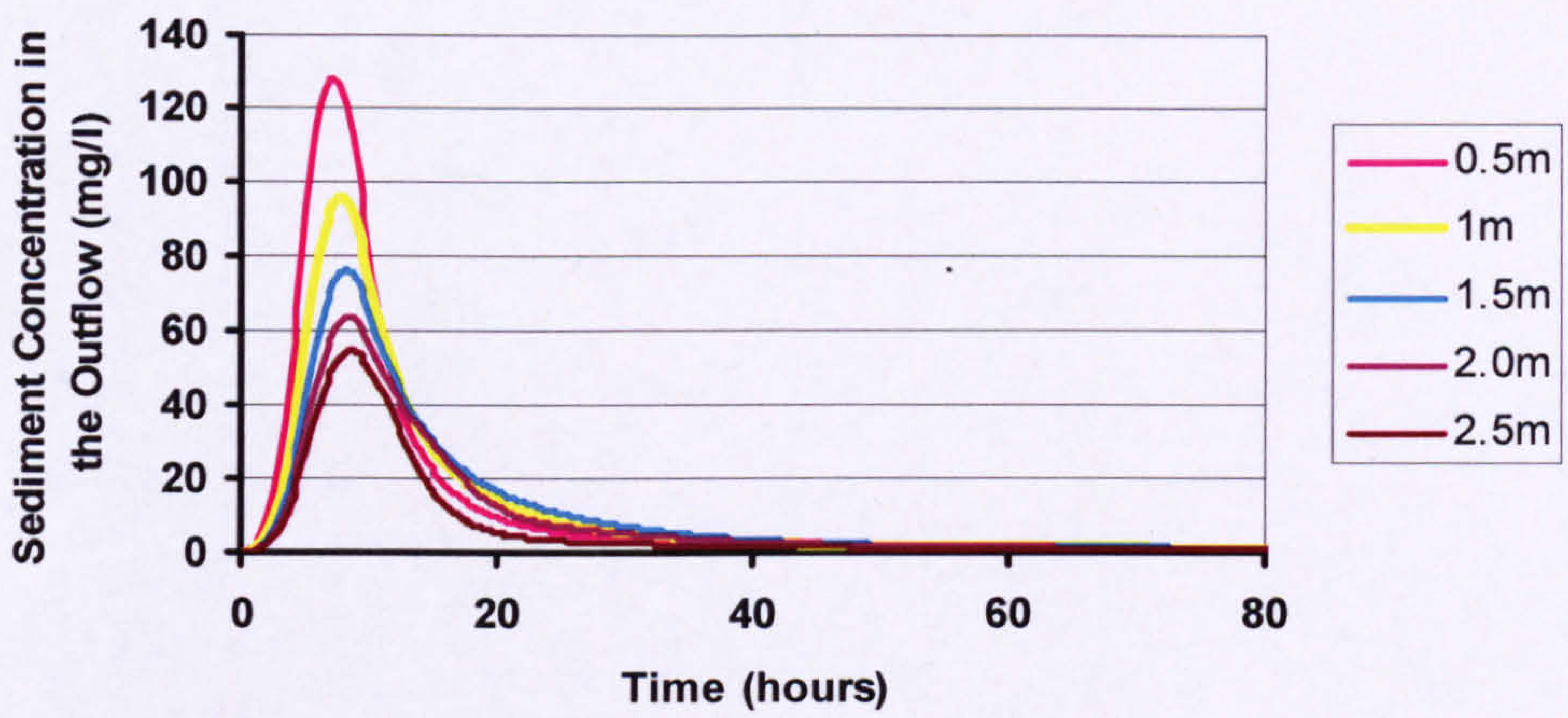


Figure 5.13: Concentration of 0.001mm diameter sediment in the outflow for five pipe elevations.

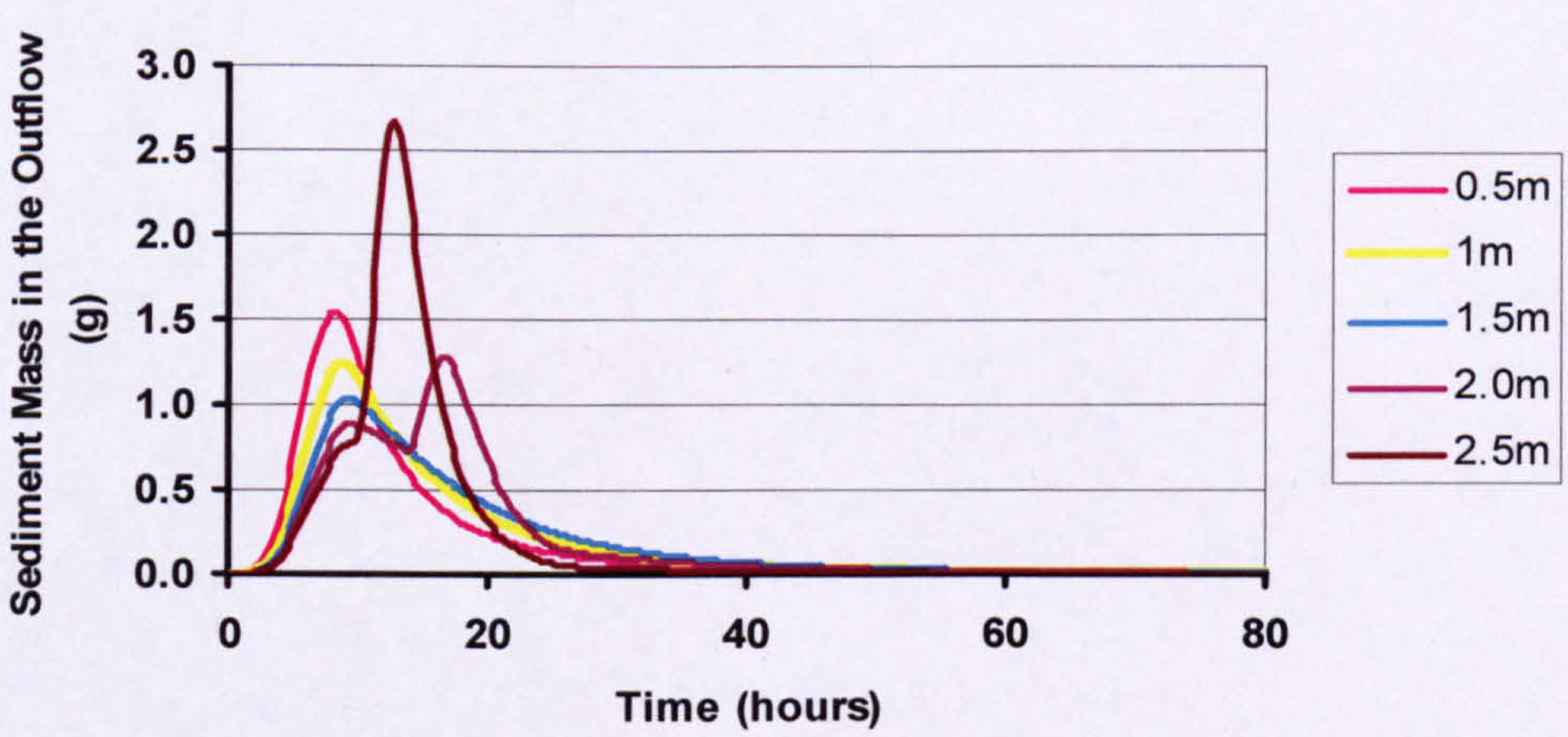


Figure 5.14: Mass of sediment in the outflow for five pipe elevations for particle size 1 (0.001mm).

Figure 5.12 shows the inflow and total outflow for the 5 different pipe elevations. The outflow hydrographs are the same for the lowest three pipe elevations because there is sufficient TSV available in the pond to enable draining to occur through the pipe only. Thus the peak outflow is the same for these cases, as shown in Table 5.4. However for the two highest pipe elevations, peak outflow increases with increasing pipe elevation, reflecting the reduction in TSV that occurs as the pipe elevation is raised closer to the weir (set at 3m). Insufficient TSV causes the water level to rise above the weir crest and the weir is brought into operation to help drain the storm. As suggested previously, and as confirmed in Table 5.4, with both outlets in operation, peak outflows are increased. The larger pond outflow occurring as a result of weir flow is exemplified by the marked decrease in minimum retention time from a maximum of 13.83 hours for the 1.5m pipe elevation (no weir flow) to 6.06 hours and 3.83 hours respectively for the 2m and 2.5m elevation cases (both weir and pipe flow).

The percentage total sediment mass settled results for the three lowest pipe elevations - where only the pipe is responsible for the drainage of the pond - show only a slight decline in performance as pipe elevation increases from 0.5 to 1.5m. Since the outflow hydrographs are the same for these three cases, the decrease in sediment settling is entirely a consequence of larger pond volumes causing greater sediment dilution. This effect is illustrated in Figure 5.13 which shows that the outflow concentration of the 0.001mm diameter sediment decreases as pipe elevation and, hence, volume increases. Similarly, in Figure 5.14 it is clear that for the first three pipe elevation cases there is a gradually decreasing peak sediment mass in the outflow due to the reduced sediment concentrations in the pond. However greater sediment masses are found in the pond outflow after about 15 hours as pipe elevation increases. Overall, these effects result in the total mass settled over the whole inflow event remaining much the same for the first three pipe elevations.

When the weir is brought into operation (pipe elevations of 2m and 2.5m) an increase in sediment mass in the outflow is clearly apparent, see Figure 5.14. Here, the positive effect of the large permanent pool volume on sediment retention (by dilution) is masked by the negative effect of large outflows, the latter causing a flushing out of sediment. This clearly illustrates the negative effect of high outflows on pond water quality performance. Overall, however, the effect of pipe elevation on sediment capture is small. It is also noted that the retention time (see, Table 5.4) does not appear to have a significant bearing on sediment capture in these cases.

Figure 5.15 compares the effect of increasing pond volume by increasing either the pond depth or the pond surface area. Increasing the pipe elevation (and IWL) provides a larger permanent pool by increasing its depth, where as by increasing the radius larger permanent pools occur as a result of increases in surface area. The comparison of the trends of sediment settled by increasing pond volume shown in Figure 5.15 using these methods indicates that sediment capture is enhanced when the pool volume is provided by increases in surface area, but is reduced when the volume increases due to increased depth. Note that in these simulations there appears to be a greater sensitivity to changes in radius than to changes in pipe elevation.

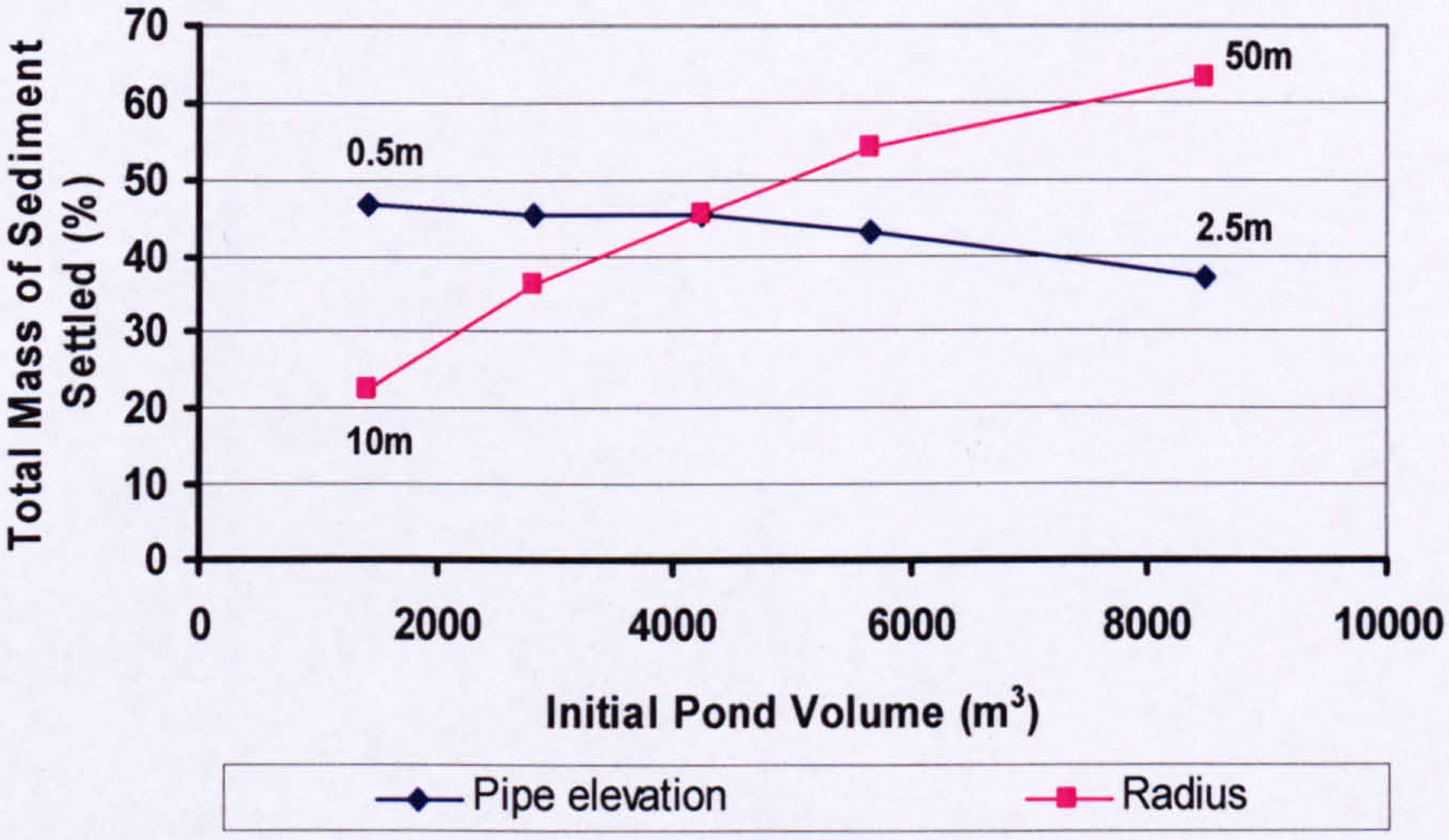


Figure 5.15: Comparison of sediment mass settled and initial pond volume when pipe elevation and radius are altered.

5.3.3 Varying Outflow Pipe Diameter

The effect of different outflow pipe diameters on water quality performance is described in this section. As discussed in Chapter 4, if a pipe is too large, with respect to the magnitude of the inflow event, often it results in much of the incoming storm water passing through the pond very quickly, resulting in a short retention time. In this analysis the range of diameters used was 0.05m-0.2m. As before, other pond parameters took the base case values.

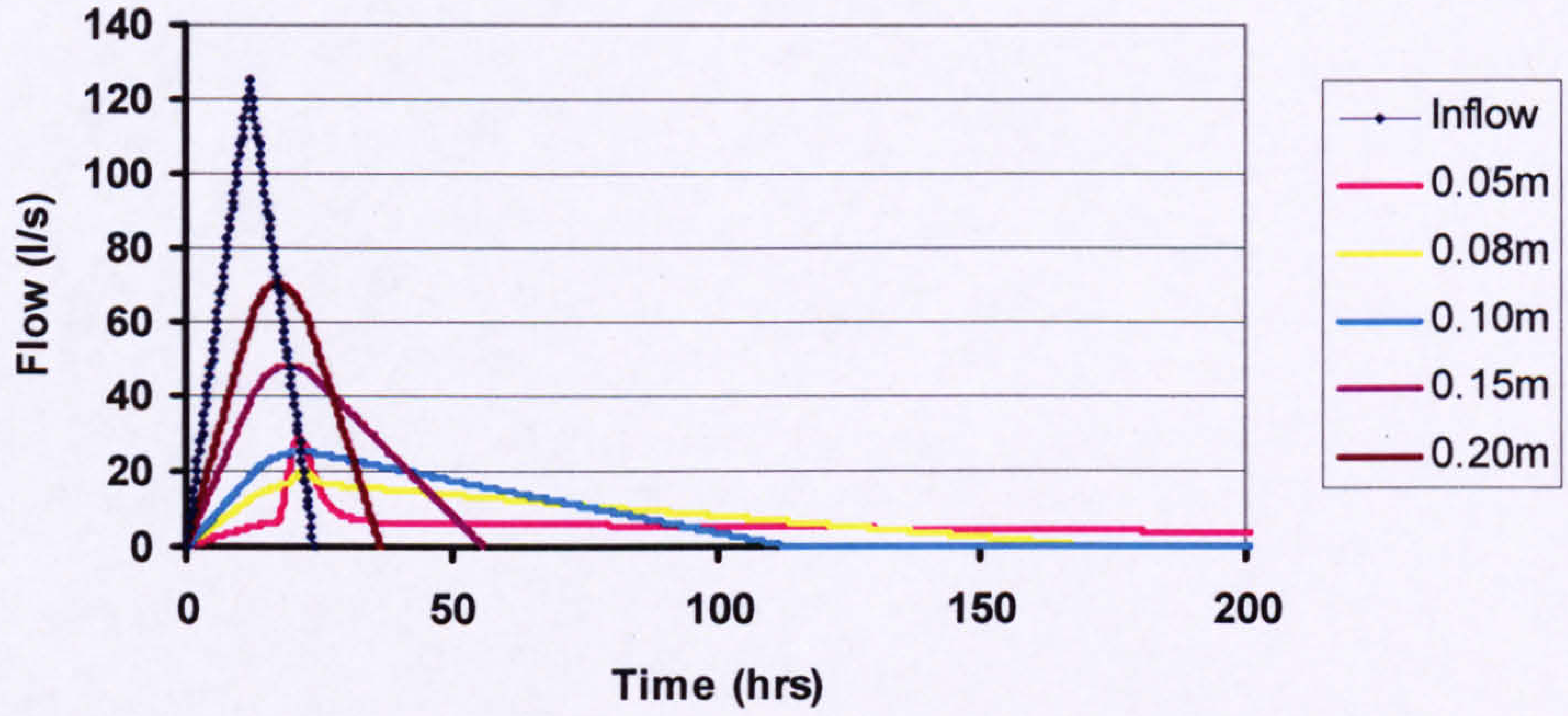


Figure 5.16: Inflow and Total Outflow for 5 pipe diameters.

Table 5.5: Flow and water quality results from ponds with different pipe diameters.

Pipe Diameter (m)	0.05	0.08	0.1	0.15	0.2
Flow					
Peak Outflow (l/s)	28.33	19.91	25.09	48.18	70.29
Peak Flow Ratio	0.23	0.16	0.20	0.38	0.56
Min Ret Time (hrs)	13.26	18.26	13.83	6.24	3.70
Mechanism of Outflow	pipe & weir	pipe & weir	pipe	pipe	pipe
Water Quality					
Total mass settled (%)	66.97	53.58	45.30	36.20	31.45
Total mass in Inflow (g)	86400	86400	86400	86400	86400
Total Mass in Outflow (g)	35767.13	49920.65	58552.16	67247.94	71249.72

Figure 5.16 shows the flows for the five different pipe diameters. In general, peak outflow increased and outflow durations decreased as pipe diameter increased because more efficient drainage occurs through larger pipes. The weir is not required to operate in the cases where larger pipes facilitate enough drainage so that water levels remain below the weir crest throughout the storm. As discussed in Chapter 4, pipes that are too small cause pond water levels to rise rapidly and hence increase the probability that the weir will be required to drain the pond. This effect occurs in the simulations with the two smallest pipe diameters. Interestingly, for these two cases the peak outflow decreases and the minimum retention time increases as the pipe diameter increases from 0.05 to 0.08m, see Table 5.5.

This occurs as a result of improved drainage through the pipe, providing a longer period before the water level reaches the crest of the weir. Retention times decrease from 13.8 to 3.7 hours for the remaining three pipe diameter cases as a result of drainage that is too efficient with respect to the size of the inflow. The settling results shown in Table 5.5 and Figure 5.17 show a clear reduction in sediment capture with increasing pipe diameter. Generally, this occurs because increasing amounts of sediment are lost in the outflow as pipe diameter increases (due to both increasing sediment concentration in the pond and increasing outflow rates).

The total sediment mass settled was highest for the two smallest pipe diameters, being 67% and 54%, respectively, despite both outlets being in operation. This demonstrates the importance of selecting the correct pipe size, with respect to the design storm, to enable a sufficient period of pond filling. It is this period of filling alongside the resultant larger pond volumes (which the two smallest pipe diameters in this analysis provided) that create the retention time required for sediment to settle out. Clearly, when the pipe is oversized retention time and sediment capture both reduce.

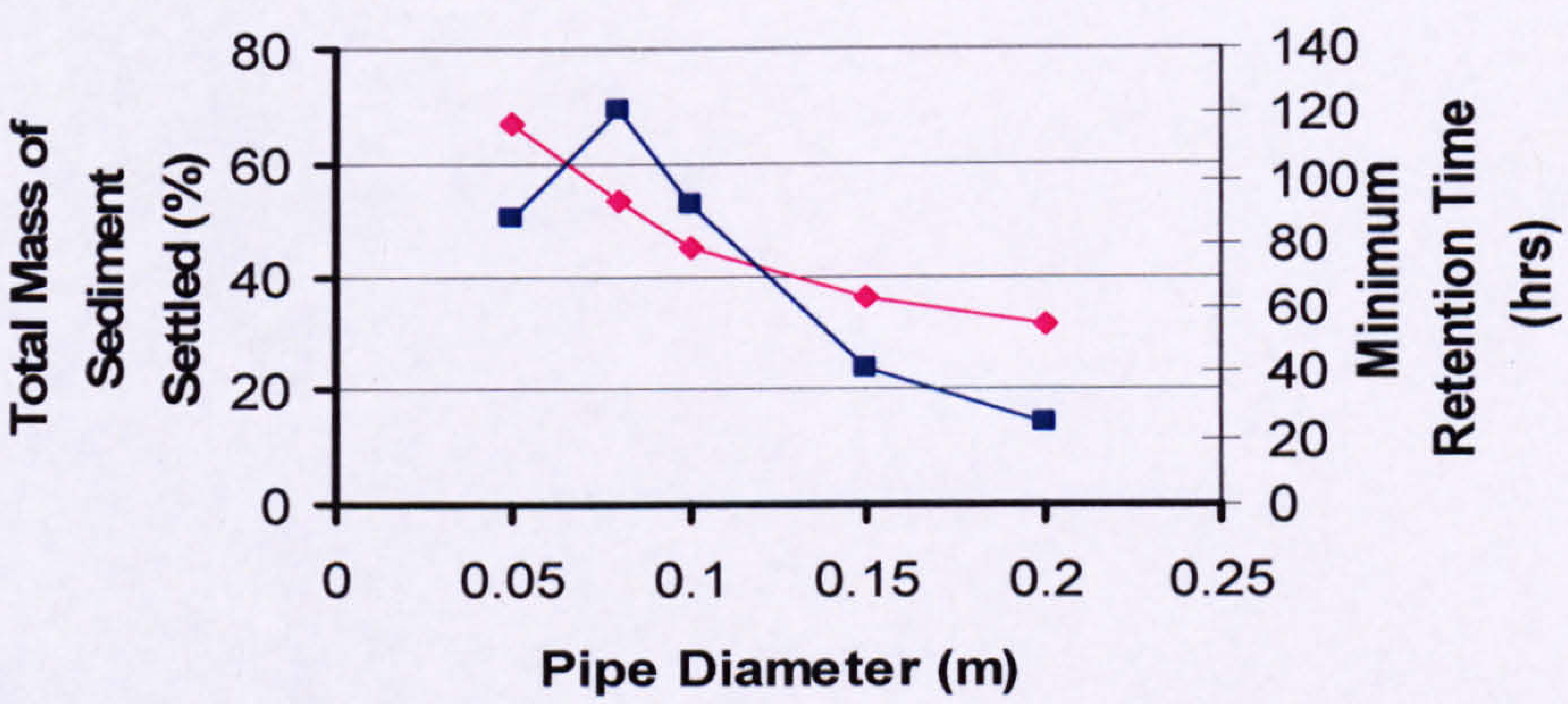


Figure 5.17 Percentage of the total mass of sediment settled with increasing pipe diameter.

5.3.4 Varying the Initial Water Level

In this sensitivity analysis a range of IWLs from 1 – 2m above the base of the pond (i.e some above and some below the outlet pipe set at 1.5m) were used to investigate the influence of IWL on pond water quality performance. In a lined retention pond IWL would not be able to drain below the level of the lowest outlet pipe elevation unless the climate enabled sufficient evaporation. In an unlined pond, there is the possibility of infiltration to groundwater. These elevations were chosen to illustrate the full effects of IWL on pond water quality performance. As before, other parameters took the base case values.

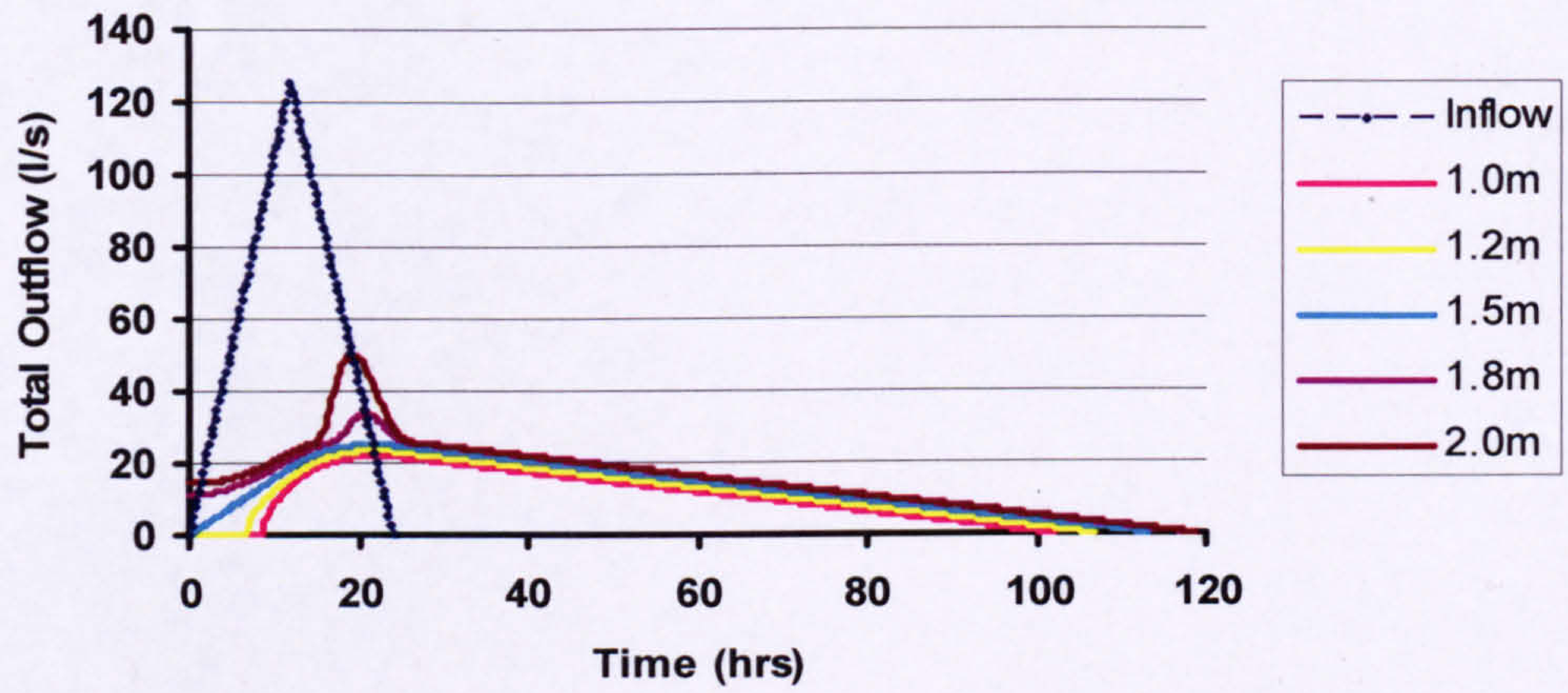


Figure 5.18: Inflow and Total Outflow for 5 Initial Water Levels.

Table 5.6: Flow and water quality results from ponds with different IWLs.

IWL (m)	1	1.2	1.5	1.8	2
Flow					
Peak Outflow (l/s)	21.89	23.30	25.09	33.70	50.02
Peak Flow Ratio	0.18	0.19	0.20	0.27	0.40
Min Retention Time (hrs)	13.99	13.88	13.83	10.91	7.52
Mechanism of outflow	pipe	pipe	pipe	Pipe & weir	Pipe & weir
Water Quality					
Total mass settled (%)	59.40	53.09	45.30	41.37	38.86
Total mass in Inflow (g)	86400	86400	86400	86400	86400
Total Mass in Outflow (g)	44313	50853	58552	62368	64861

Figure 5.18 and Table 5.6 show that as IWL increases, peak outflows increase as does the duration of the outflow. This is simply due to the decreasing TSV available at the start of the storm as the IWL is increased. The duration of the outflow increases with increasing initial water elevation because of the greater volume of water requiring discharge from the pond. In the two cases where the initial water level is below the elevation of the outlet

pipe (1.5m), outflow does not start until the pond water level has risen to the elevation of the pipe. In contrast, for the three cases where the initial water level is equal to or above the elevation of the outlet pipe, outflow begins immediately at the start of the simulation.

Figure 5.19 shows that the minimum retention time decreased as IWL increased. In the simulations where only the pipe is operating (IWLs 1 –1.5m), a marginal reduction in minimum retention time occurred as IWL increased. This reflects slightly larger outflows due to higher heads over the pipe. However the decrease in retention time was much more apparent in the two highest IWL cases, where both the weir and the pipe operate. This occurs because the reduction in TSV is such that the weir is used to drain the pond in all of the cases where the IWL is greater than 1.5m. With both outlets draining the pond, retention times are reduced from 13.8 – 7.5 hours.

The mass of sediment settling decreased with increasing IWL (Table 5.6) due to the effect of increasing pond volume (and its diluting effect on pond sediment concentration), higher outlet heads and the operation of the weir. The increase in the pond volume that occurs with increasing IWL, causes a decrease in sediment concentration which would be expected to improve pond performance as a result of the reduction in sediment mass in the outflow. However in these simulations, higher pond water levels provide higher heads that increase the rate of pond drainage even before the weir operates, thereby increasing sediment mass lost in the outflow, and hence reducing the mass of sediment settling in the pond. Here, as in section 5.3.2, the effect of sediment being flushed out of the outflow as a consequence of inadequate TSV masks any improvement in sediment capture that might occur from the increase in sediment dilution caused by larger pond volume.

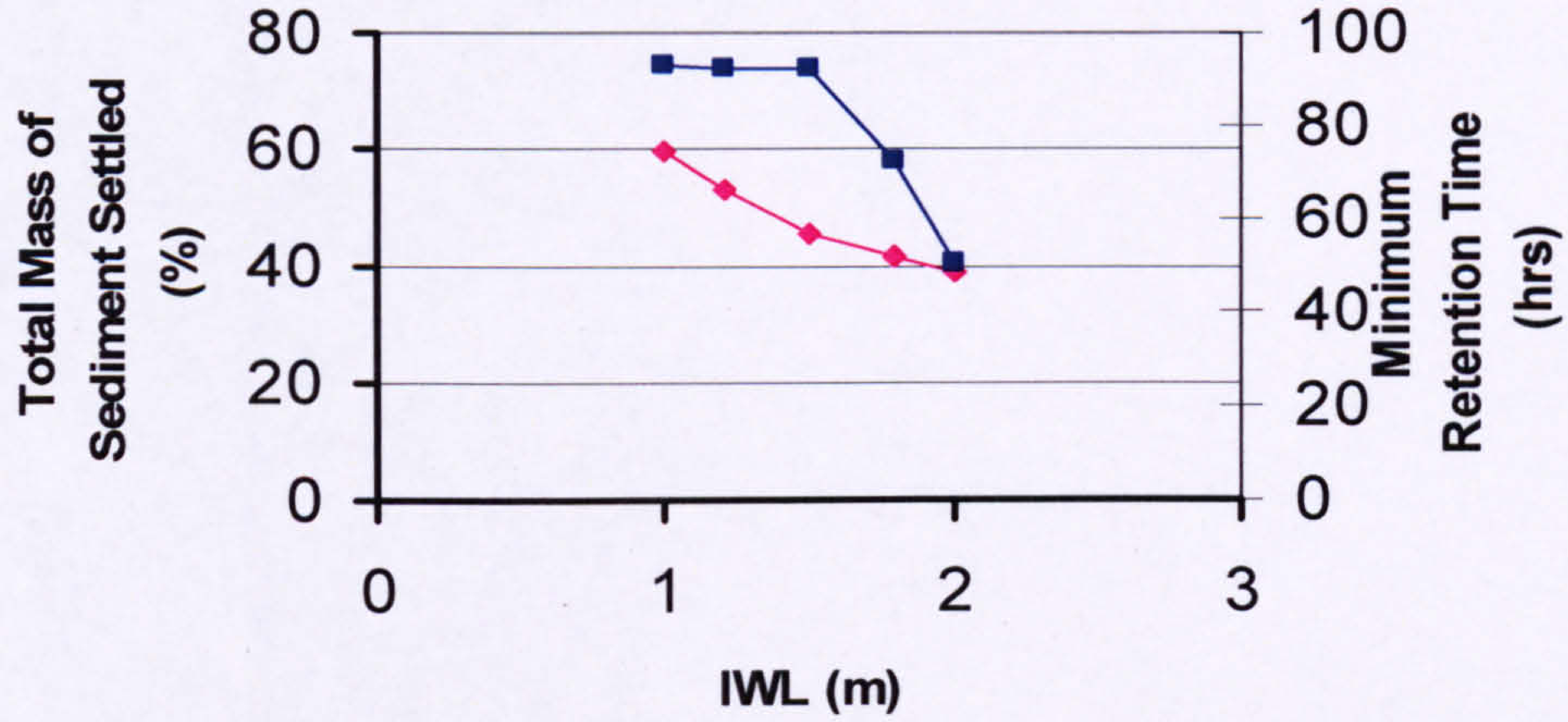


Figure 5.19: Percentage of the total mass of sediment settled with increasing Initial Water Level.

5.3.5 Varying the Sediment Inflow Duration

Finally, the effect of sediment inflow duration on pond water quality performance was considered because the current method of designing ponds for water quality treatment in the UK assumes that the majority of sediment enters the pond in the runoff generated by the first 12-15mm of a storm, which means that sediment inflow duration will vary depending on the size of the storm. The analysis conducted here, considers sediment inflow durations ranging from 7.2 hours – 24 hours of a 24 hour duration storm. All the other parameters were fixed at the base case values. Since the sediment inflow duration is the only parameter being altered, the outflow and retention time results are the same for all the cases.

Table 5.7: : Flow and water quality results from ponds with different Sediment Inflow Durations.

Sediment inflow duration (hrs)	**7.2**	**9.6**	**11.52**	**15.36**	**19.2**	**24**
Flow						
Peak Outflow (l/s)	22.10	22.10	22.10	22.10	22.10	22.10
Peak Flow Ratio	0.22	0.22	0.22	0.22	0.22	0.22
Min Retention Time (hrs)	2.094	2.094	2.094	2.094	2.094	2.094
Water Quality						
Total mass settled (%)	47.16	45.30	44.15	42.60	41.83	41.39
Total mass in Inflow (g)	86400	86400	86400	86400	86400	86400
Total Mass in Outflow (g)	56544.02	58552.16	59796.24	61467.47	62305.71	62781.01

Results from Table 5.7 show that the total mass settled decreases marginally with increasing sediment inflow duration. Since the sediment mass in the inflow (and all other pond parameters) remain constant, this can only be a reflection of the timing of the peak sediment concentration relative to the peak of the pond outflow. Figure 5.20 shows the concentration of sediment in the outflow for the shortest (7.2 hours) and longest (24 hours) sediment inflow durations, together with the pond outflow. It clearly shows that when the bulk of the sediment is washed into the pond before the peak of the outflow, sediment concentrations in the pond outflow remain high (the shortest duration case). However when the peak sediment concentration coincides more closely with the peak outflow, sediment concentration in the outflow is much more dilute and less sediment is washed out with the outflow. This effect is also apparent from Figure 5.21, where it is clear that most of the sediment mass has been flushed out before the peak of the outflow for the 7.2 hour sediment duration event, while for the 24 hour sediment duration event more of the sediment mass is flushed out at a later time. Overall, however, the total sediment mass flushed out (the areas under the curves in Figure 5.21) is similar. Hence the sediment capture performance is only slightly reduced as the sediment inflow duration is increased.

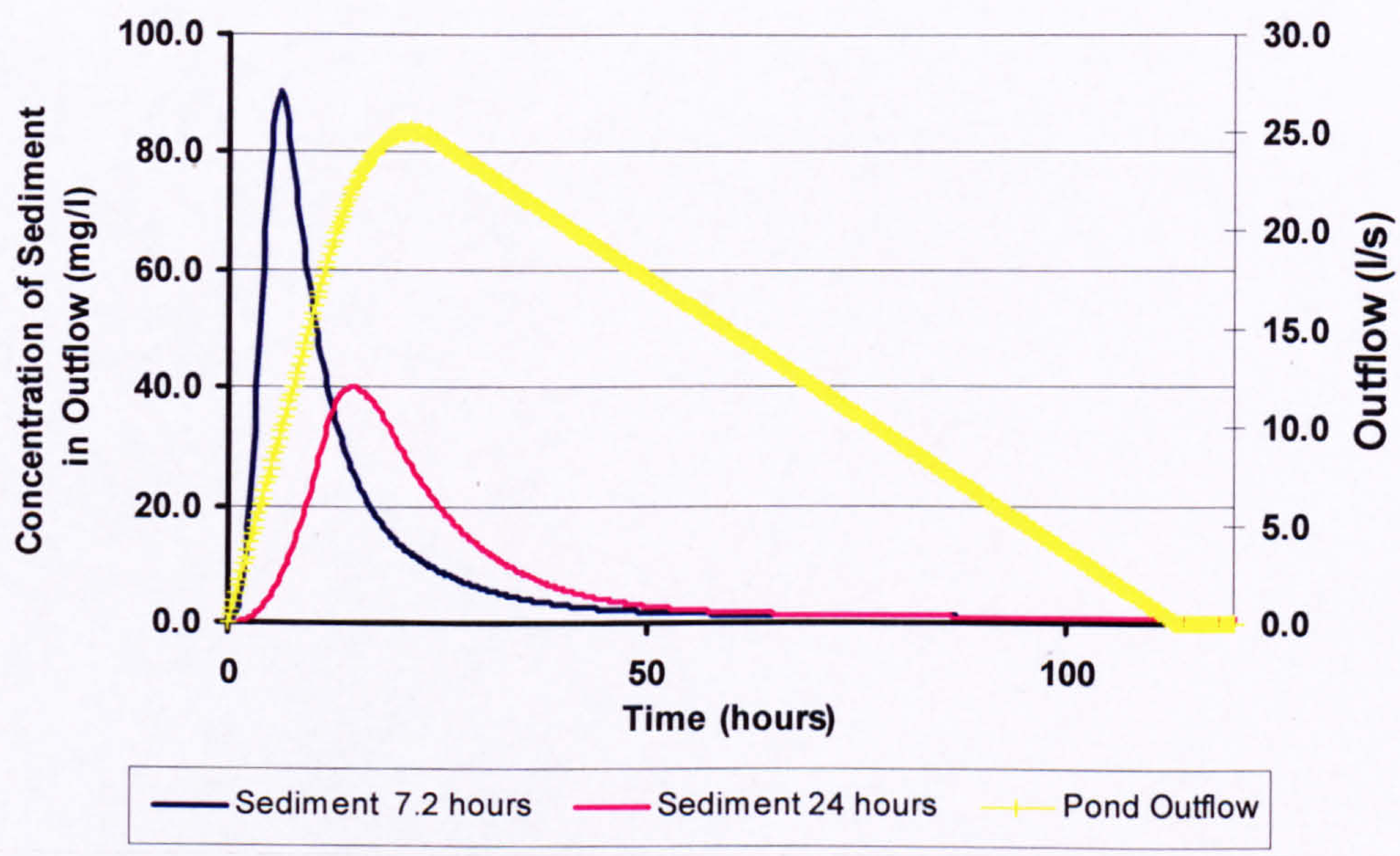


Figure 5.20: Concentration of sediment in the outflow for the 7.2 and 24 hour sediment duration inflow events. Pond outflow is shown in yellow for reference.

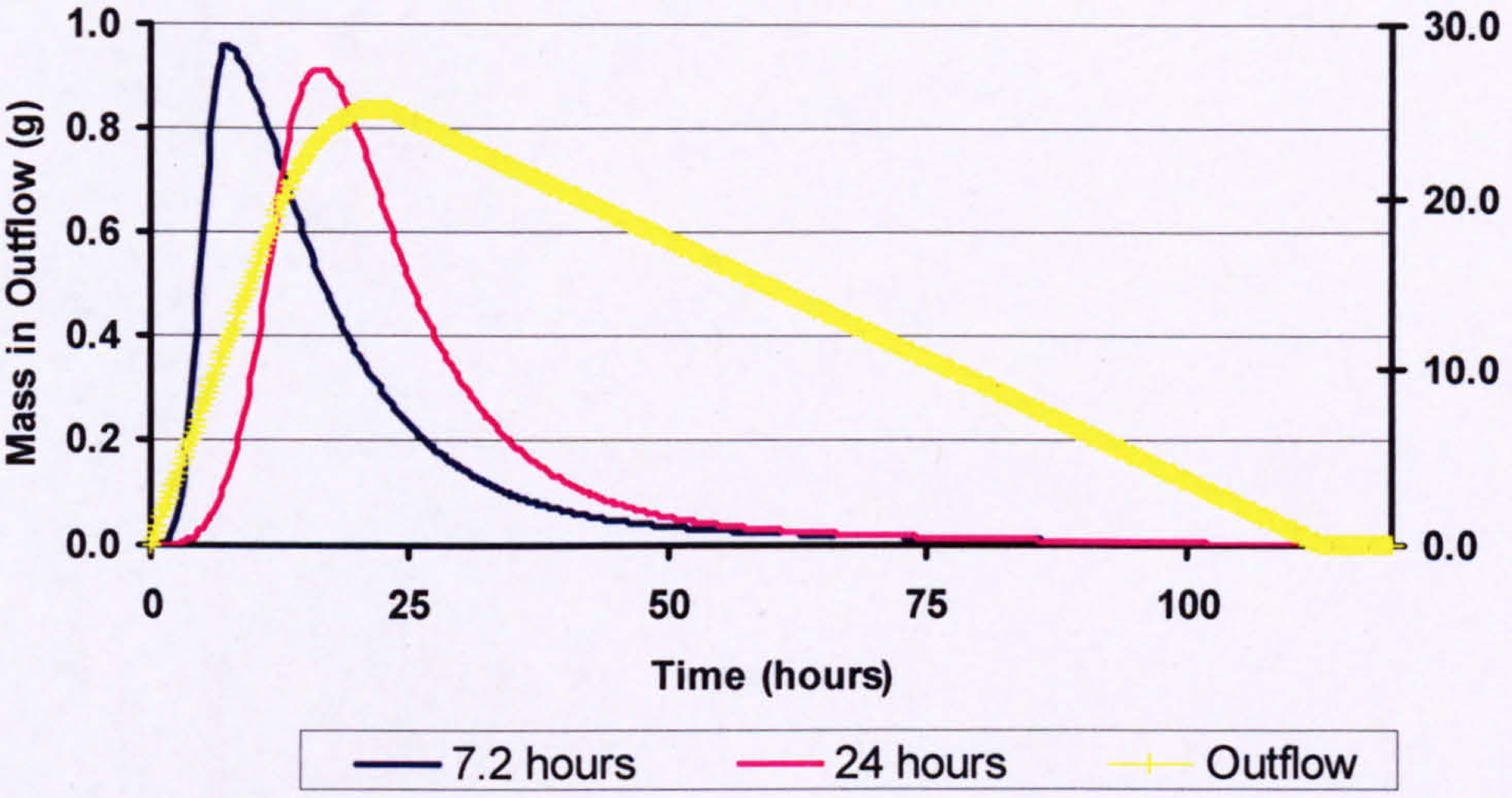


Figure 5.21: Mass of sediment in the outflow for the 7.2 and 24 hour sediment duration inflow events. Pond outflow is shown in yellow for reference.

5.3.6 Summary

In general, the sediment capture performance of retention ponds is affected in the following ways. Sediment capture increases due to increases in pond radius, but decreases by increases in outlet pipe diameter and by increases in initial water level. Smaller decreases in performance occur due to increases in outlet pipe elevation and even smaller decreases occur due to increases in sediment inflow duration.

Overall, the ability of retention ponds to capture sediment is intrinsically linked to the mechanisms of in-pond dilution and outlet operation. The former is controlled by the pond volume whilst the latter is influenced by the water level in the pond (controlling the head driven pipe outflow) and also by the availability of TSV (determining whether or not weir outflow occurs).

Large pond volumes act to reduce in-pond sediment concentration by dilution, thus reducing the mass of sediment lost through the outlet(s). Furthermore, comparing ponds with different radii and different pipe elevations showed that increases in surface area enhanced sediment capture but increases in depth reduced sediment capture. Ponds with insufficient TSV inevitably required the weir to operate, causing a large flushing out of sediment. Similarly, pond configurations that maximised TSV tended to capture more sediment.

Clearly, however, there are some competing issues here. For example, a balance has to be achieved between providing enough TSV in a pond to maximise the volume of the storm runoff captured before weir flow occurs while providing a large enough permanent pool to encourage sediment settling. The analysis conducted here has shown that large shallow ponds are the best way of providing a permanent pool and simultaneously enabling TSV to be maximised (since in this way large pool volumes can be provided with relatively low water levels), but this is not ideal in terms of the consumption of valuable urban land.

The literature suggests that retention time is a major influence on pond sediment capture, and that sediment removal increases with longer retention times. However the sensitivity analysis simulations showed that in some cases sediment capture varied inconsistently with retention time. Analysis of all the base case pond simulations (Figure 5.5) found a positive correlation between the minimum retention time and the percentage total sediment mass settled. Thus whilst retention time affects the sediment mass retained, it is not the only influence on the sediment capture performance of retention ponds.

Results from the sensitivity analysis indicated that pond design is crucial in achieving water quality performance targets. In general the pond simulations that were most efficient in retaining sediment mass were those with the largest permanent pool volumes. The operation of the weir increased the mass of sediment in the outflow in all cases, reducing the water quality performance of the pond. It has already been shown in Chapter 4 that the design of the outlet is important for flow control and here it has been demonstrated that it also plays a critical role in pond water quality performance, in terms of the sediment capture.

6 Pond Design and Water Quality Performance

Since ponds are designed differently depending on whether they are targeted at meeting flow or water quality standards, this chapter investigates how both flow and water quality performance are affected when:

(1) The pond is designed primarily to meet flow attenuation standards

(2) The pond is designed primarily to meet water quality standards

The results from this analysis were then applied to analyse sediment retention performance at Linburn Pond. All simulations reported were undertaken using the two-zone water quality model described in Chapter 5. As in the previous chapter, input parameters such as the particle size distribution, volume ratio and transfer coefficient were kept constant. Parameters governing pond geometry, volume and outlet configuration were varied to investigate the effect of pond design on water quality performance.

As discussed in Chapter 2, sediments are the most influential non-point source of pollutants to surface waters by mass. They provide the binding surface to which pollutants, heavy metals and hydrocarbons adhere, and once in the water environment, these pollutants can persist for long periods of time [*Deletic*, 1998a; *Delleur*, 2001]. The retention of sediment in a pond, by the mechanism of settling, reduces the concentration of sediments in water leaving the pond and hence reduces the pollutant load entering downstream water courses. In this chapter, the degree of sediment settling or sediment capture in the pond was used as a measure to quantify water quality performance.

6.1 Pond Design for Water Quality Enhancement

The important factors considered when designing a pond were discussed in full in Chapter 4, with respect to pond design for flow attenuation. Here a brief discussion of the important considerations for pond design for sediment capture is presented.

Stormwater basins are designed to provide water quality benefits by reducing the concentration of polluting material present in stormwater to pre-development levels. This includes suspended sediment, trace metals, nutrients, hydrocarbons such as oil and grease, and pesticides as well as any litter and debris entering the basin. Retention ponds improve water quality by a number of processes. These may be physical, by means of sedimentation, chemical through the process of precipitation and biological by the uptake of nutrients and microbial degradation [*Heal and Drain*, 2003; *Novotny*, 1994]

The high rate of removal of these pollutants is highly dependant upon pond design. According to the United States Highways Administration, factors that affect pond water quality performance include the residence time, the depth of the permanent pool, the existence of a plunge pool, the ratio of catchment area to permanent pool surface area, and the presence and type of aquatic vegetation [*Federal Highways Administration*, 2004].

In retention ponds, the settling of sediment is primarily dependant upon the characteristics of the permanent pool. For example, a larger permanent pool enhances particulate settling by increasing residence time, and may also provide conditions for growth of aquatic vegetation, thereby enhancing filtration, nutrient uptake and creating conditions suitable for microbial degradation. Detention basins, however, are typically dry between storms since they are designed to provide flow control rather than water quality treatment and hence there is no permanent pool. However since detention basins temporarily detain stormwater, some water quality enhancement may take place in a shallow temporary pool.

6.2 Water Quality Performance and Design Standards

Just as with standards for flow attenuation, water quality performance standards for ponds vary internationally and regionally. According to [*Pitt*, 2005], retention ponds have been extensively monitored under a variety of conditions and, if designed properly and well maintained, can achieve suspended solids removal rates of between 70-90%. In order to ensure that maximum sediment removal occurs, the pond must be sized properly. Table 6.1 shows the estimated pond surface areas (as a % of catchment area) required for different land uses to remove sediment particles with diameters greater than 5μm and 20μm which correspond to 90% and 65% reduction in the mass of suspended solids, respectively.

The [*Alberta Drainage Design Bulletin*, 2003] suggests that all BMPs (including wet ponds) should provide a minimum of 85% removal of sediments of diameter 75μm and larger prior to discharge into natural drainage courses. According to the [*New Jersey*

Department of Watershed Management, 2003], a new development that creates more than 0.1 hectares of impervious surface must provide stormwater management tools that reduce the average annual load of total suspended solids by 80%. Much of the design literature is concerned with sizing the pond correctly to ensure adequate retention times [*CIRIA*, 2000; *France*, 2002; *Tennessee BMP Guidelines*, 2003], while other design guidance focuses on capturing the first flush [*Mappleridge BMP Guidelines*, 2001; *Tennessee BMP Guidelines*, 2003]. Since there are currently no agreed water quality objectives in the UK, in the analysis reported in this chapter a water quality target of 80% was used in accordance with much of the design guidance used in the United States and Canada.

In the simulations conducted here, this target is achieved when there is an 80% reduction in the mass of sediment in the outflow compared to the mass of sediment in the inflow (i.e. the pond has removed 80% of the incoming sediment mass)

Table 6.1: Pond surface area requirements (as a % of the catchment area) for different land uses to remove particles with diameter greater than 5μm and 20μm adapted from [*Pitt*, 2005],

Landuse	5μm	20 μm
Totally paved	3.0	1.1
Highways	2.8	1.0
Industrial	2.0	0.8
Commercial	1.7	0.6
Institutional	1.7	0.6
Residential	0.8	0.3
Open spaces	0.6	0.2
Construction site	1.5	0.5

In the USA, the permanent pool volume is often defined as the volume equivalent to three times the water quality volume (WQV). WQV is equivalent to 12.7 mm of runoff from the contributing drainage area, representing the 'first flush' of a storm. This has led to requirements in some states to capture and treat the first 12.7mm of a storm. In other states, the WQV of a storm is defined as the first 25.4 mm (1 inch) of runoff from the impervious area in a catchment. In the USA in areas defined as ultra-urban, stormwater quality targets are frequently limited to treating only the WQV of a storm event [*Federal Highways Administration*, 2004].

In the UK the treatment volume (Vt) is the primary consideration in the design of ponds for water quality enhancement. The treatment volume is defined by [*CIRIA*, 2000] as the volume that contains the most polluted part of the runoff from a storm (i.e. the first flush). In order for a pond to treat this part of the runoff, [*CIRIA*, 2000] advise that a pond should have a permanent pool of volume 4Vt (Table 6.2). However, recent research suggests that this volume is larger than necessary for some types of runoff (e.g. residential) and that a permanent pool volume of just one Vt may be adequate enough to ensure that the most polluted part of the runoff is treated [*McLean et al.*, 2005].

Although pollutant reduction occurs by a number of mechanisms in retention ponds, in this chapter water quality enhancement by means of sedimentation is the only process considered, since this mechanism is the most significant process in water quality improvement in ponds [*Pitt*, 2005].

6.3 Pond Configuration and Flow Attenuation Design

This section presents the results of simulations that were conducted to investigate how three different pond configurations perform for water quality improvement (in terms of removing sediment from stormwater inflow). The three pond configurations are described in full below. All three ponds were designed to meet the flow attenuation criteria, using the flow model as described in Chapter 4, where ponds should reduce the peak outflow to 50% (or less) of the peak inflow for a given storm. As in Chapter 4, the design storm magnitude used was the 1 in 25 year event of 250l/s peak flow with a duration of 24 hours.

Configuration 1: A Typical Detention Basin

Configuration 1 (Figure 6.1) is a typical detention basin. The radius of the basin was sized to 26.9m in order to meet the flow attenuation criteria, as described above. As for all the pond configurations in this chapter, the 90° V-notch weir crest was set at 3m above the base of the pond, in keeping with UK design guidance on health and safety. Since detention basins typically do not have a permanent pool, the pipe outlet of 0.1m diameter was set at an elevation of 0m above the base to allow complete draining of the basin after a storm. This type of basin was selected for study since there is a growing body of evidence to suggest that detention basins may remove a significant fraction of the polluting load from stormwater runoff [*Mead*, 2003; *Sniffer*, 2002; *United States Evironmental Protection*

Agency, 1987]. The simulations conducted in this section enable the performance of such a basin to be quantified under various conditions, and allow direct comparisons with other basin types.

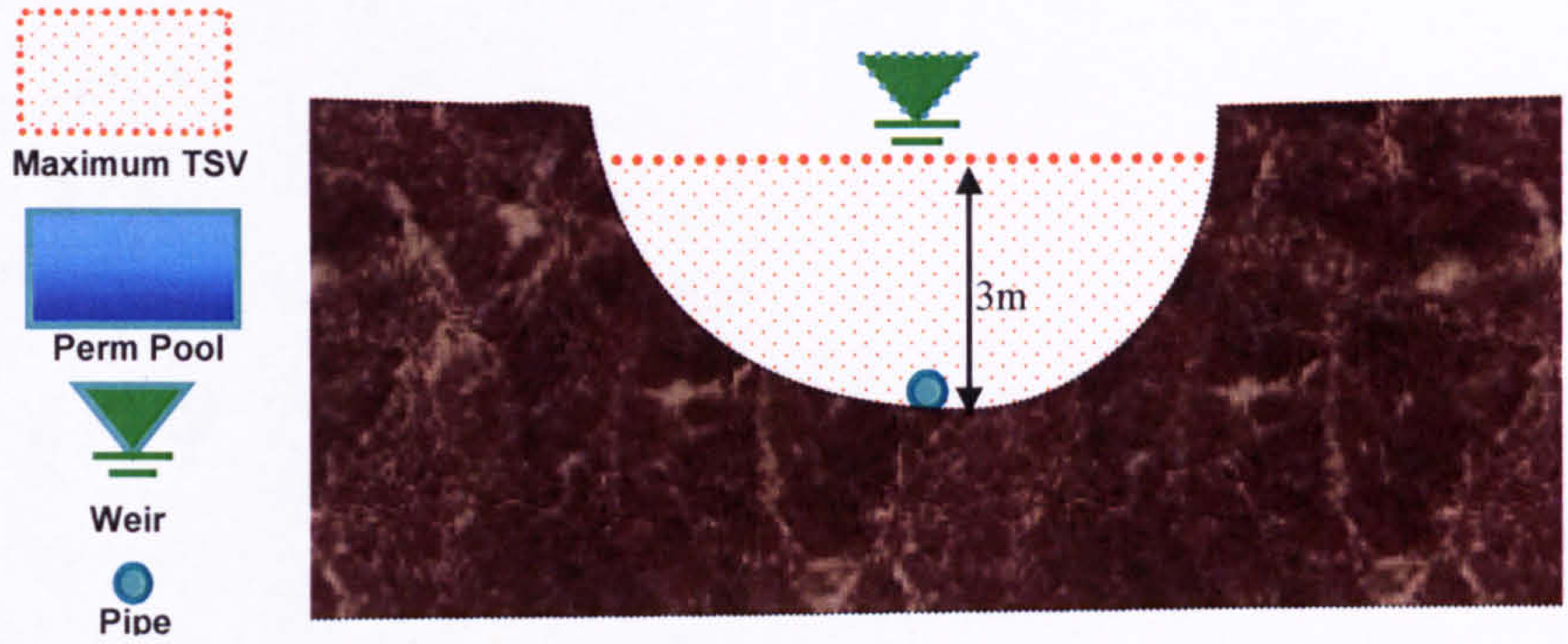


Figure 6.1: Configuration 1 - the typical detention basin (schematic diagram not to scale). Note that the basin starts empty, and the pipe is at an elevation of 0m in the basin to enable immediate drainage of storm water. The weir crest is set at an elevation of 3m.

Configuration 2: A Single Outlet Retention Pond

Configuration 2 (Figure 6.2) is an example of one type of retention pond that has been built in the UK to date, e.g. Linburn Pond. Configuration 2 was designed to have a single outlet only, a 90° v-notch weir, which is set at 3m above the base of the pond. This configuration has a permanent pool, however since there is only a single high-level outlet, the pool is prevented from draining below the weir crest, and thus the pond is continually full, with no temporary storage volume for incoming stormwater. In these simulations the pond had a radius of 75m in order to meet the flow criteria.

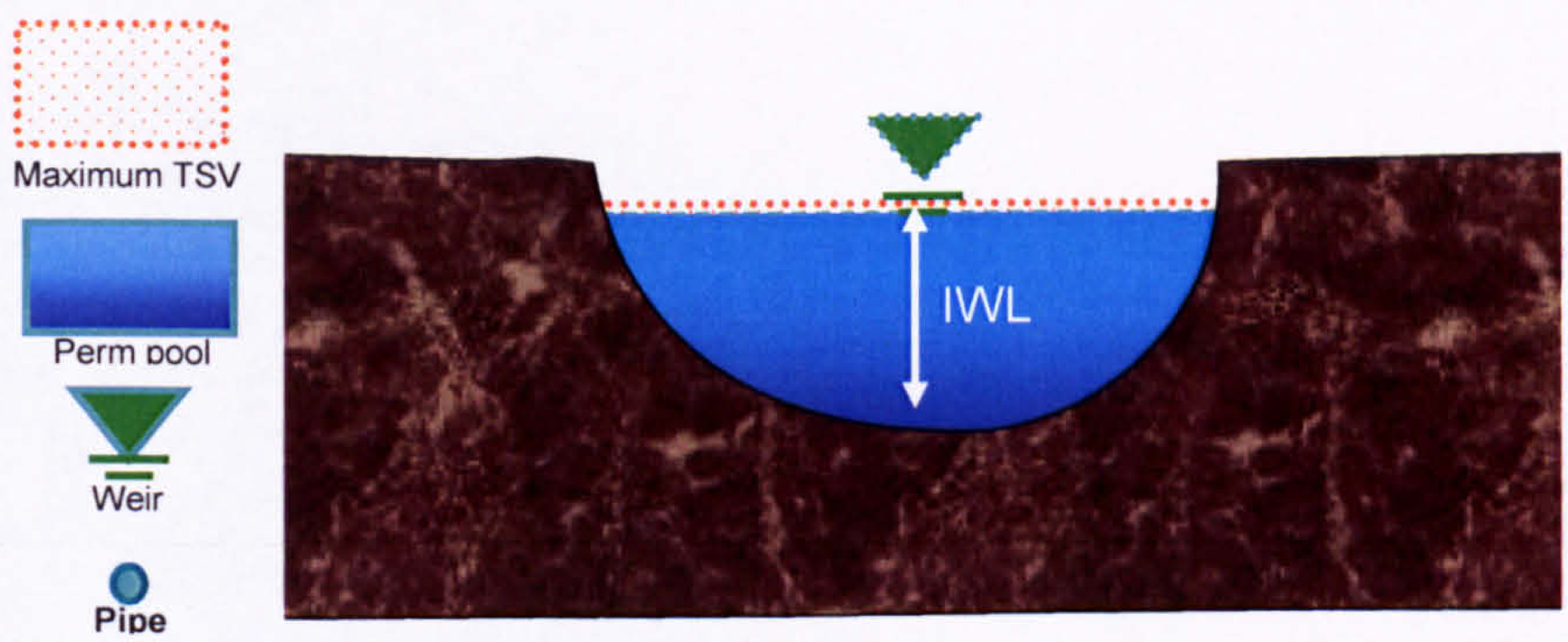


Figure 6.2: Configuration 2 a single-level outlet retention pond (Schematic diagram not to scale). Note the lack of a pipe in this configuration results in the pond being permanently full to the crest of the weir (as indicated by the green triangle). Also note that under such conditions there is no available TSV.

Configuration 3: A dual outlet retention pond

Pond configuration 3 (Figure 6.3) is an example of an ideal retention pond (with multi-level outlets) as used in the sensitivity analyses conducted in section 5.3 The pond was sized to have a radius of 36.2m in order to meet the flow criteria for the design storm event. The pond has a dual outlet with a 90° v-notch weir crest set at 3m above the base and a pipe of 0.1m diameter set at 1.5m above the base of the pond to enable a permanent pool to remain after draining.

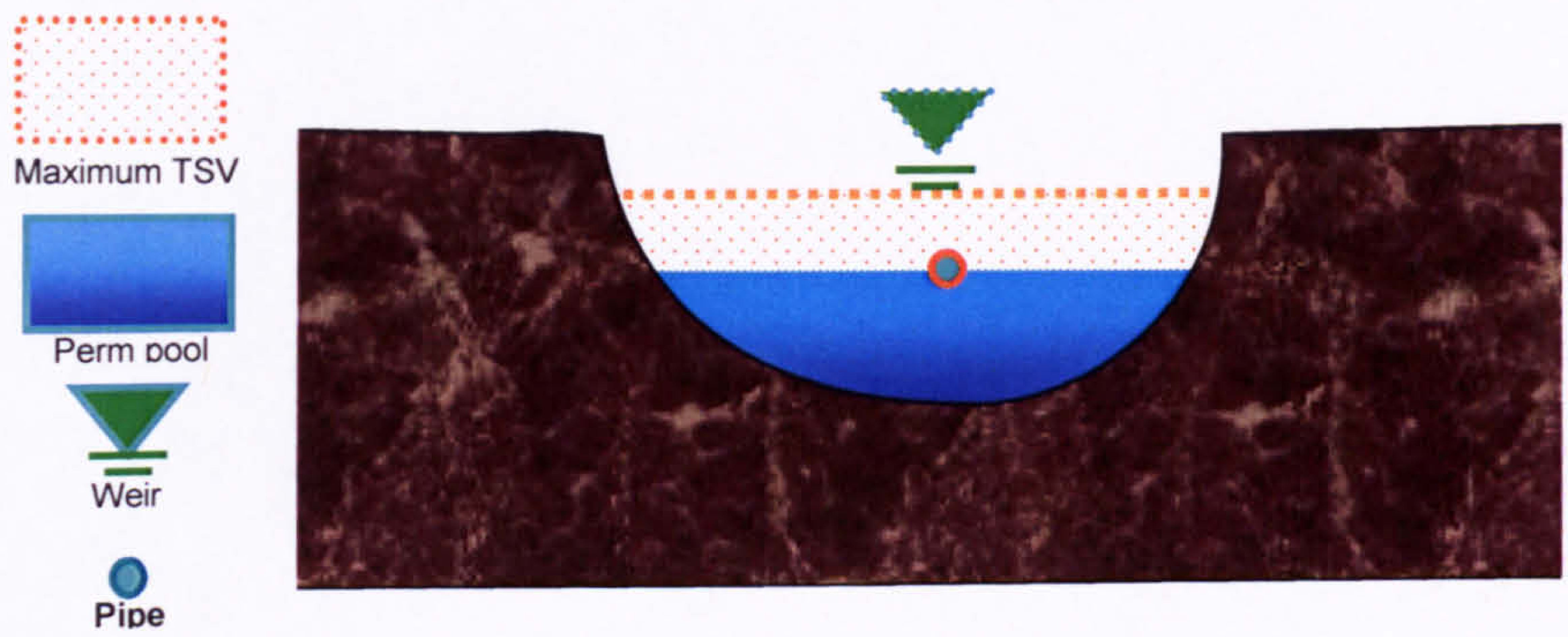


Figure 6.3: Configuration 3, a dual-level outlet retention pond (Schematic diagram not to scale). Note the presence of the secondary outlet at an elevation of 1.5m in the pond enables the presence of a permanent pool, whilst also enabling the provision of TSV (as indicated by the dotted area).

6.3.1 Performance in the 1 in 2 year event

Chapter 4, introduced a pond design procedure in which a pond was sized to meet the flow attenuation criteria for a large design storm, and then tested to determine how it performs for smaller storms of a higher frequency. While Chapter 4 examined the flow attenuation performance of ponds designed in this manner, this section investigates the water quality performance of ponds designed solely for flow control.

All three pond configurations were sized as discussed above to meet the flow attenuation criteria for the 1 in 25 year storm event. Using the two-zone sediment model (as described in section 5.1) a 1 in 2 year storm of 125l/s peak flow with a duration of 24 hours was then run through the pond to determine the effect of the flow design on the water quality performance of the three pond configurations. The storm carried a sediment load of 86400g, which was distributed over 9.6 hours (the first 15mm of runoff – as suggested in [*CIRIA*, 2000] in designing the pond to meet Vt guidelines), peaking at 100mg/l (Figure

6.4), as in the base case pond in the sensitivity analysis described in Section 5.2. Results are reported in Section 6.3.3 below. A typical model output is shown in Figure 6.4, showing pond flows and sediment inflow and outflow for the dual outlet pond, Configuration 3, for a 1 in 2 year event.

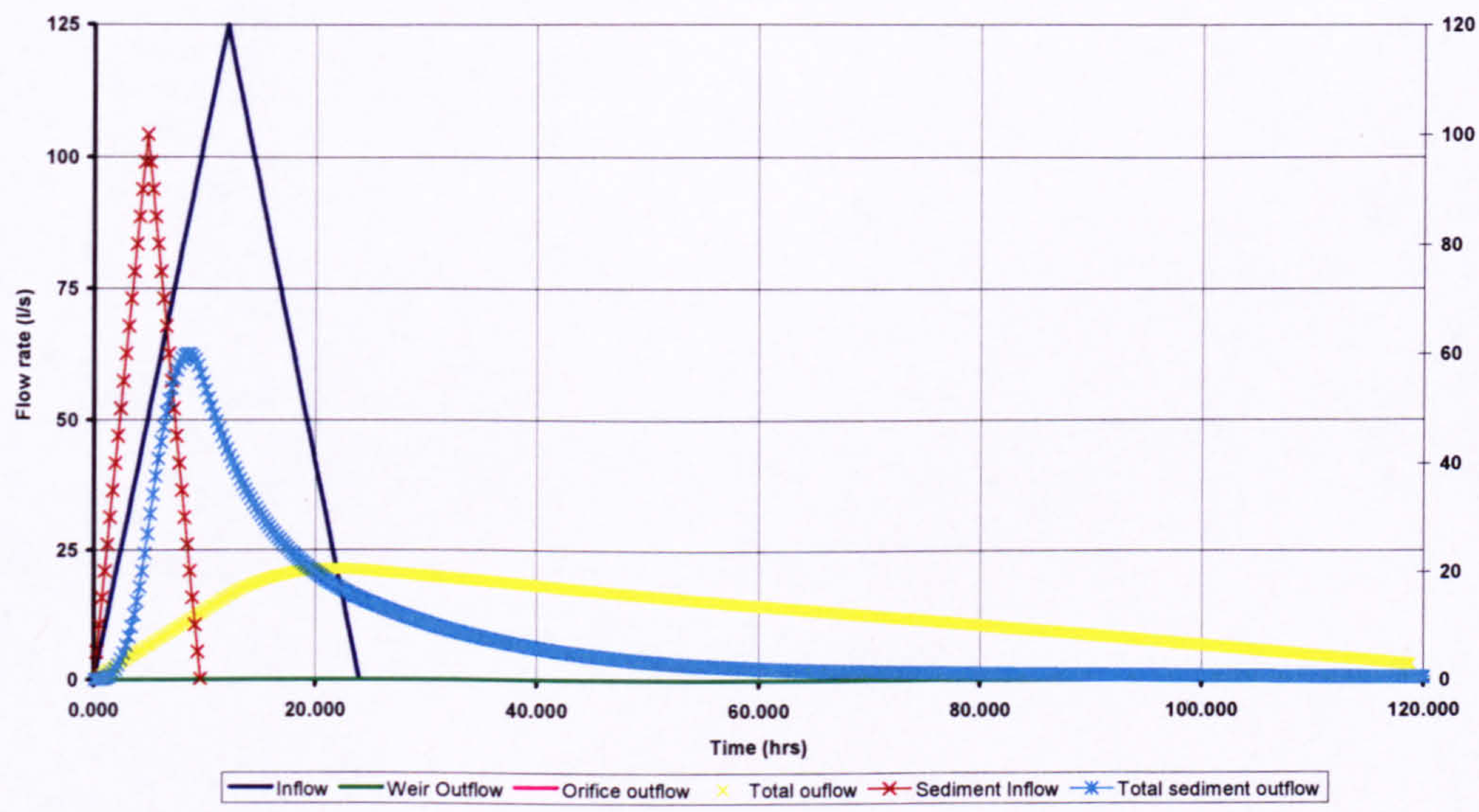


Figure 6.4: Storm inflow and outflow and sediment inflow and total sediment outflow in pond configuration 3 running a 1 in 2 year event.

6.3.2 Performance in the Q90 storm event

The same three pond configurations were used to investigate water quality performance under even smaller, but more frequent storm conditions, which according to much of the literature, are more critical events for pond water quality [*Campbell et al.*, 2004; *McLean et al.*, 2005]. Research suggests that smaller more frequent storms are important in terms of the pollutant load carried into the pond. [*McLean et al.*, 2005] suggested that stormwater ponds should be designed to provide effective treatment for 90% of storms occurring annually, since these more frequent storms are more influential for pond water quality, than the large extreme events which are more detrimental to the pond's flow attenuation performance. For this reason, a 30 year daily rainfall data set from Tullyallan in East Scotland were analysed to produce the 24 hour duration Q90 event. The rainfall data were obtained from the BADC, and was previously used in Chapter 4 to provide a long-term inflow series. The daily Q90 event was found to have a peak flow of 28.7l/s suggesting that 90% of the events occurring in the Tullyallan catchment have a peak flow of at least

28.7l/s, and therefore the pond's performance for this storm would be indicative of how the pond might perform on a daily basis.

6.3.3 Results

Table 6.3 shows the main results for flow and water quality for the three pond configurations. Figure 6.5 illustrates the percentage total mass of sediment settled for each pond configuration for the 1 in 2 year storm event, while Figure 6.6 shows the percentage total mass of sediment settled for each pond configuration for the Q90 event.

Table 6.2:Performance of 3 pond configurations designed to meet flow design criteria for a small storm (Q90): peak flow reduction and total mass of sediment settled

	Configuration 1	Configuration 2	Configuration 3
Flow (% peak flow reduction)			
Permanent pool volume (m^3)	0	53014	6175
1 in 2	78	69	83
Q90	60	94	68
Water Quality (total mass settled %)			
1 in 2	47	78	50
Q90	52	98	70

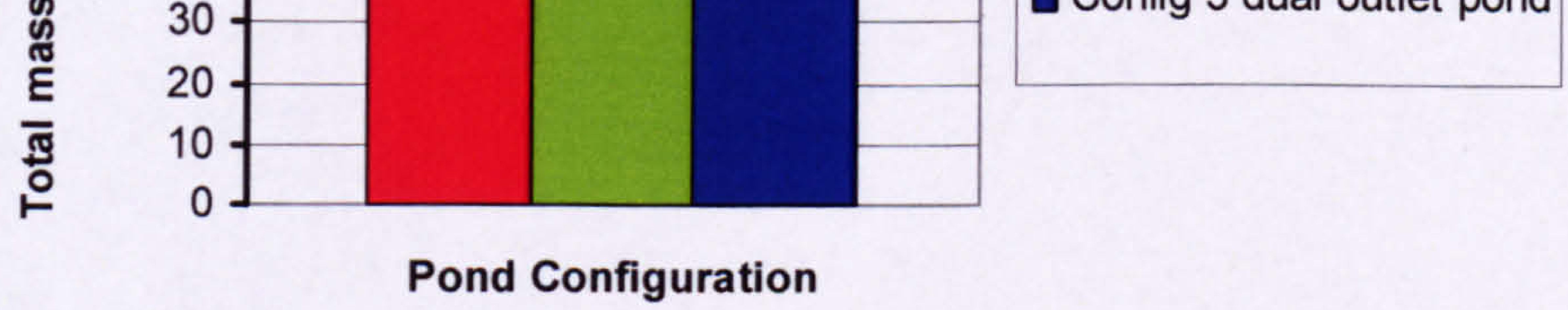


Figure 6.5: Water quality performance (in terms of total mass of sediment settled) of three pond configurations designed to meet flow attenuation design criteria for the 1 in 2 storm

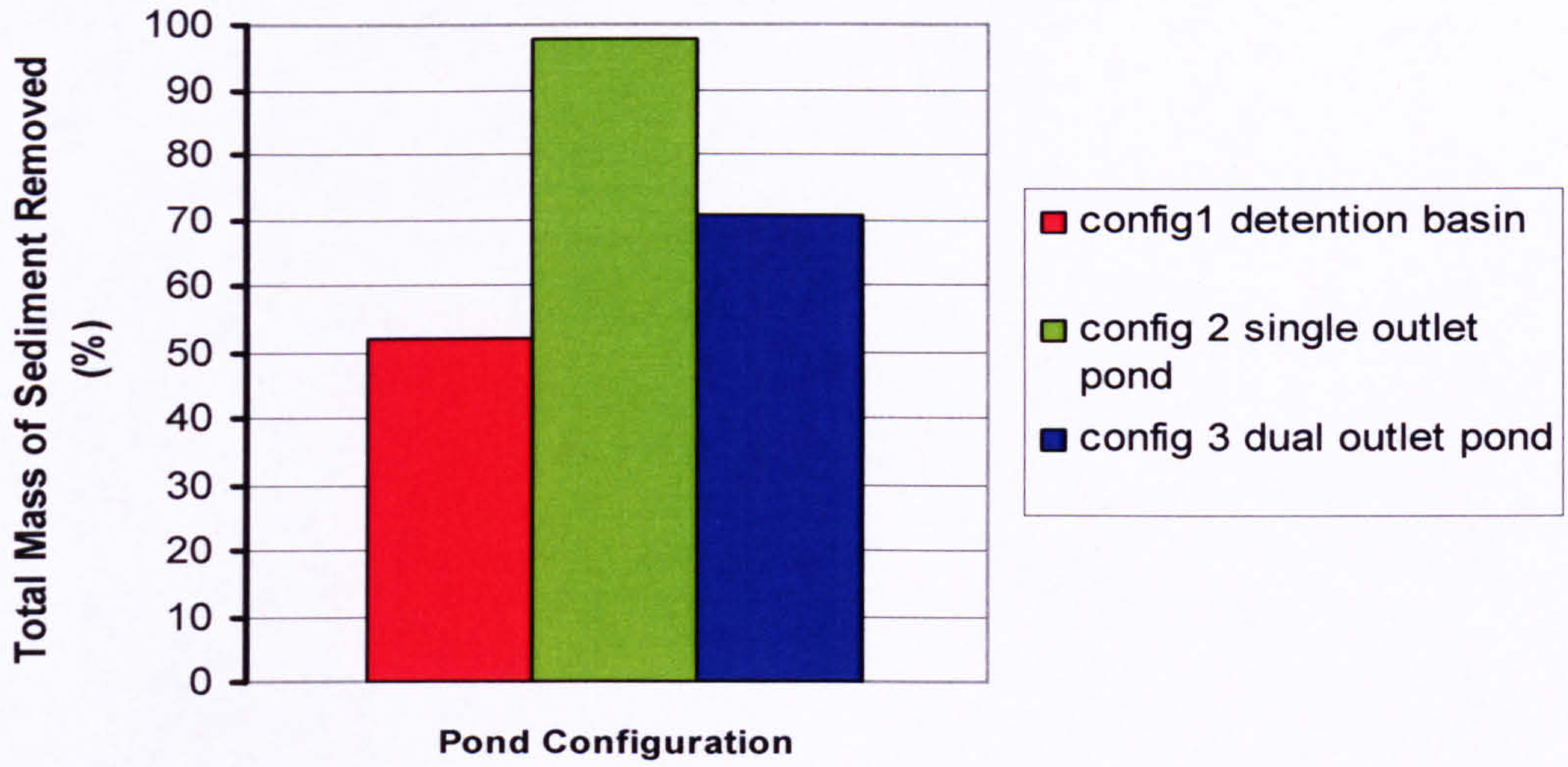


Figure 6.6: Water Quality performance of 3 pond configurations designed to meet flow attenuation design criteria for a small (Q90) storm

It is clear from Table 6.3 that all three ponds meet the flow criteria for the 1 in 2 year storm event (since they have been designed to reduce the 1 in 25 year flow event by 50%). The dual-level outlet retention pond, Configuration 3 has the best flow attenuation performance, with the highest peak flow reduction. Both configurations 1 and 3 (dual outlet ponds) are able to attenuate the storm and drain fully without overflow via the weir. In terms of water quality performance, the single outlet pond, Configuration 2 was most effective at reducing sediment mass, (78% compared to 47% for Configuration 1 and 50% for Configuration 3, Figure 6.5). This is entirely due to Configuration 2 having a large permanent pool volume; 53014m^3 compared to 6175m^3 and 0m^3 for configurations 3 and 1 respectively. The large pool volume produces much lower sediment outflow concentrations in the pond, as the incoming sediment mass is immediately diluted. Despite this, none of the ponds meet the 80% sediment removal target.

The results in Table 6.3 also show how the 3 pond configurations perform for the Q90 event. Since this is the smallest (and in reality would be the most frequent) of the storms simulated, it is the critical event size for water quality assessment. Clearly, all three configurations substantially reduce the peak flow rate, with the highest reduction of 94% occurring in the single-level outlet pond, Configuration 2. This pond also has the longest retention time, and removes the most sediment (98%), passing the sediment removal criteria set out earlier in the chapter. The excellent performance of this pond in both flow and sediment attenuation is a result of this configuration having over four times the surface area and permanent pool volume of the other two ponds. The detention basin, Configuration 1, performed least well of the three configurations both in terms of flow and water quality with a peak flow reduction of 60% and a sediment removal of 52%. This configuration also had the shortest retention time. In the UK, detention basins are not recognised as providing a significant contribution to water quality, however the findings of this work are typical of research conducted internationally as shown in Table 2.2 in Chapter 2. However, it should be noted that despite having no permanent pool volume, water levels in the detention basin rose to 0.3m during the course of the storm providing time (and sufficient depth and volume) for over half of the sediment mass to settle out. Configuration 3 attenuated flows and removed a significant portion (70%) of the sediment load. Its performance for the Q90 event indicated that a pond designed to 'just' meet the flow attenuation target might remove around 70% of the incoming sediment on a daily basis – despite having a permanent pool over 8 times smaller than that of the best performing pond, Configuration 2.

Water quality performance for all three pond configurations was better under smaller storm magnitudes (Q90) than in the 1 in 2 year events. Since it is these small storms which frequently wash polluting material into water courses, they are the critical event size for stormwater management. The demonstration in these results that ponds designed for flow attenuation show the greatest improvements for small, critical events suggest that flow attenuation should be considered when designing for water quality performance, as well as treatment volumes. The results also suggest that relatively small changes in pond design may produce significant improvements in pond performance for both flow and sediment attenuation.

6.4 Pond Configuration and Water Quality Design

The previous section investigated how designing a pond purely to meet flow attenuation criteria affected pond water quality performance. This section uses suggested UK design guidance for water quality to determine the pond size required for each of the three outlet configurations in Figures 6.1-6.3. The UK guidelines are based on sizing the pond to contain a multiple of the treatment volume (Vt) as defined in [*CIRIA*, 2000], as previously discussed in Section 6.2. Using the same storm magnitudes as in Section 6.3, here, the effect of designing ponds to meet water quality (as opposed to flow attenuation) criteria on pond water quality performance for the same three pond configurations is examined.

Design guidelines in the UK previously suggested that the permanent pool volume of a retention pond should be sized to contain 4 times the treatment volume (4Vt) (Table 6.2). However, recent research has resulted in new criteria suggested by [*McLean et al.*, 2005], whereby the permanent pool volume of a retention pond should be sized to contain just Vt. In this section, all three ponds were sized to contain the new proposed volume of one Vt. This volume was calculated using guidance from [*CIRIA*, 2000] as being the runoff volume associated with the first 15mm of a storm event. Because of the differing outlet configurations, the radii required to contain Vt were not the same for all three pond configurations.

The catchment area used to determine Vt is defined in Appendix C and was previously employed to estimate the size of the 1 in 2 and Q90 events in Chapter 4. Using this catchment area, the first 15mm of rainfall results in a Vt of 2550 m^3 assuming a 10% impervious urban area in the catchment. An example calculation of Vt can be found in Appendix D.

The pond radius required to retain the treatment volume for configurations 1 and 2 is the same. This is because Vt is considered to reside below the lowest outlet. In both the detention basin (Configuration 1) and the single outlet retention pond (Configuration 2) the weir, set at an elevation of 3m, is considered to be the lowest outlet (since the other outlet in the detention basin is set at 0m – below which the treatment volume could not reside and is therefore not considered in the calculation). Thus both these basins have the same radius. The dual outlet retention pond (Configuration 3) requires a larger radius to contain the same treatment volume, as its lowest outlet is at an elevation of 1.5 m above the base of the pond (as opposed to 3 m). The radius for Configuration 3 is required to be 23.26m in comparison to 16.45 m for both Configurations 1 and 2.

6.4.1 Performance in a 1 in 2 year and a Q90 storm event

To determine the effect of designing ponds to retain Vt, the same two storms described in Section 6.3 (a 1 in 2 year storm with a peak of 125l/s and a duration of 24 hours and the daily Q90 event, determined to have a peak flow of 28.7l/s) were routed through the three pond configurations. Figure 6.7 shows a typical model output with sediment inflow and outflow concentrations and the concentration of sediment in each zone during a 1 in 2 year flow event in the dual-outlet pond, Configuration 3.

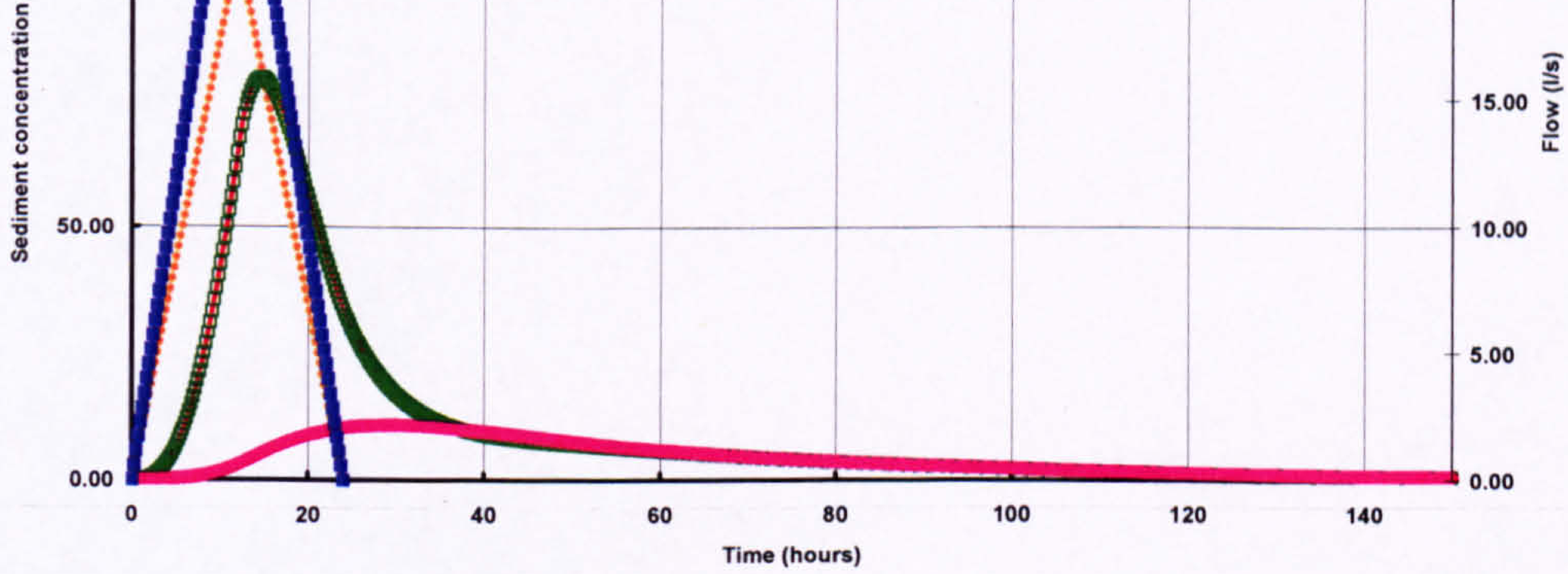


Figure 6.7: Stormwater inflow and sediment concentrations in the inflow, outflow, transport zone, 1 and the quiescent zone, 2 for the Q90 event in Configuration 3 (dual outlet pond).

6.4.2 Results

Table 6.4 and Figures 6.8 and 6.9 illustrate how the three pond configurations attenuate flows and improve water quality when the ponds have been designed to meet the suggested water quality UK design guidance. In Table 6.9, data shown in red font indicate a peak flow reduction of less than 50% (i.e. failure to meet the flow attenuation criterion).

Table 6.3: Flow and water quality performance for three pond configurations for the 1 in 2 and Q90 events when designed for water quality. Although Configuration 1 has no permanent pool, *denotes the potential pool volume beneath the weir.

	Pond Configuration		
	Configuration 1	Configuration 2	Configuration 3
Permanent pool volume (m3)	0 (*2550)	2550	2550
	Flow (Peak Flow Reduction %)		
1 in 2	32	2.0	35
Q90	45	4.0	56
	Water Quality (total mass settled %)		
1 in 2	23	21	29
Q90	37	48	55

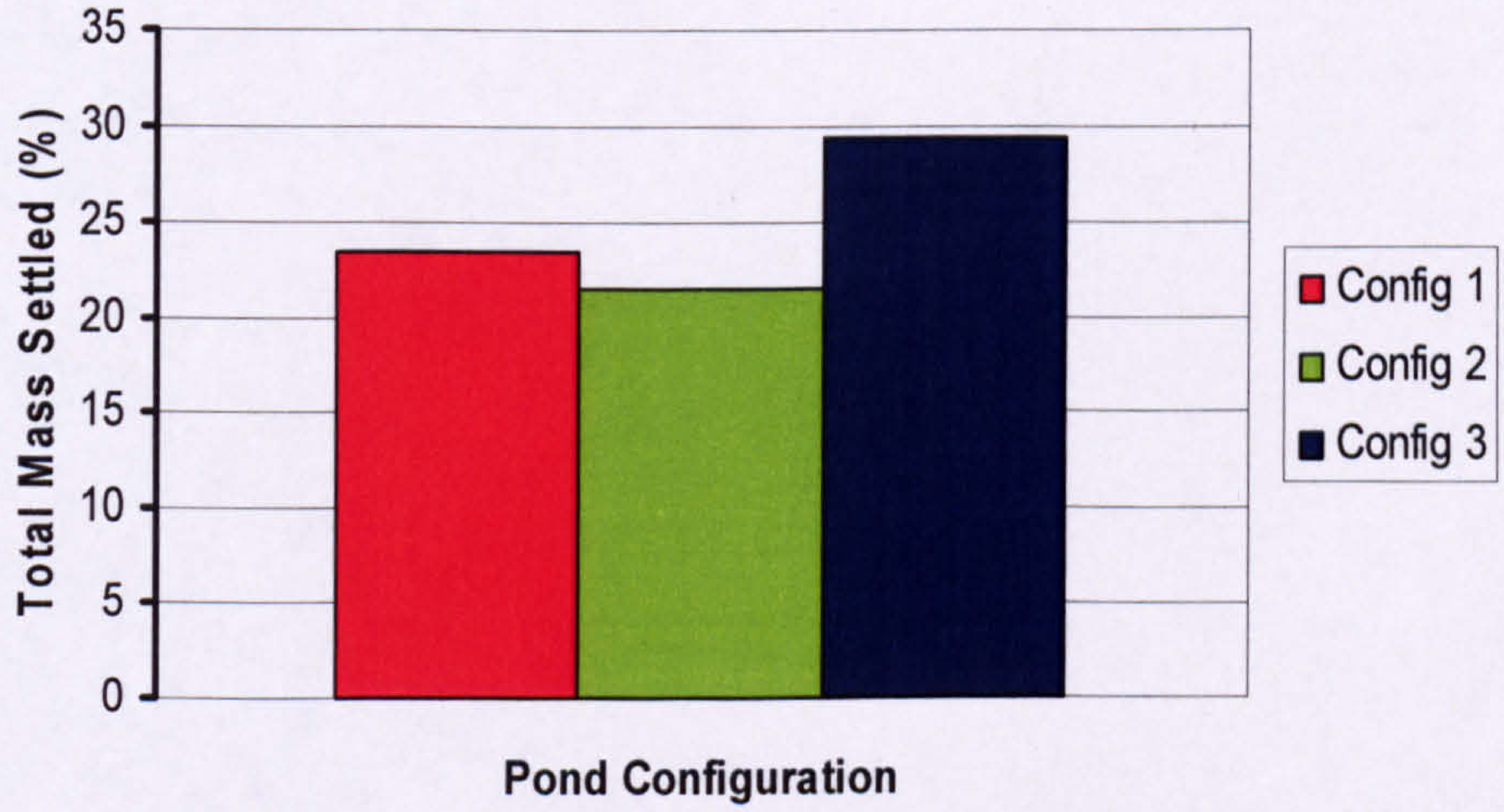


Figure 6.8: Water Quality Performance of 3 pond configurations designed to meet water quality criteria, following a 1 in 2 year storm event

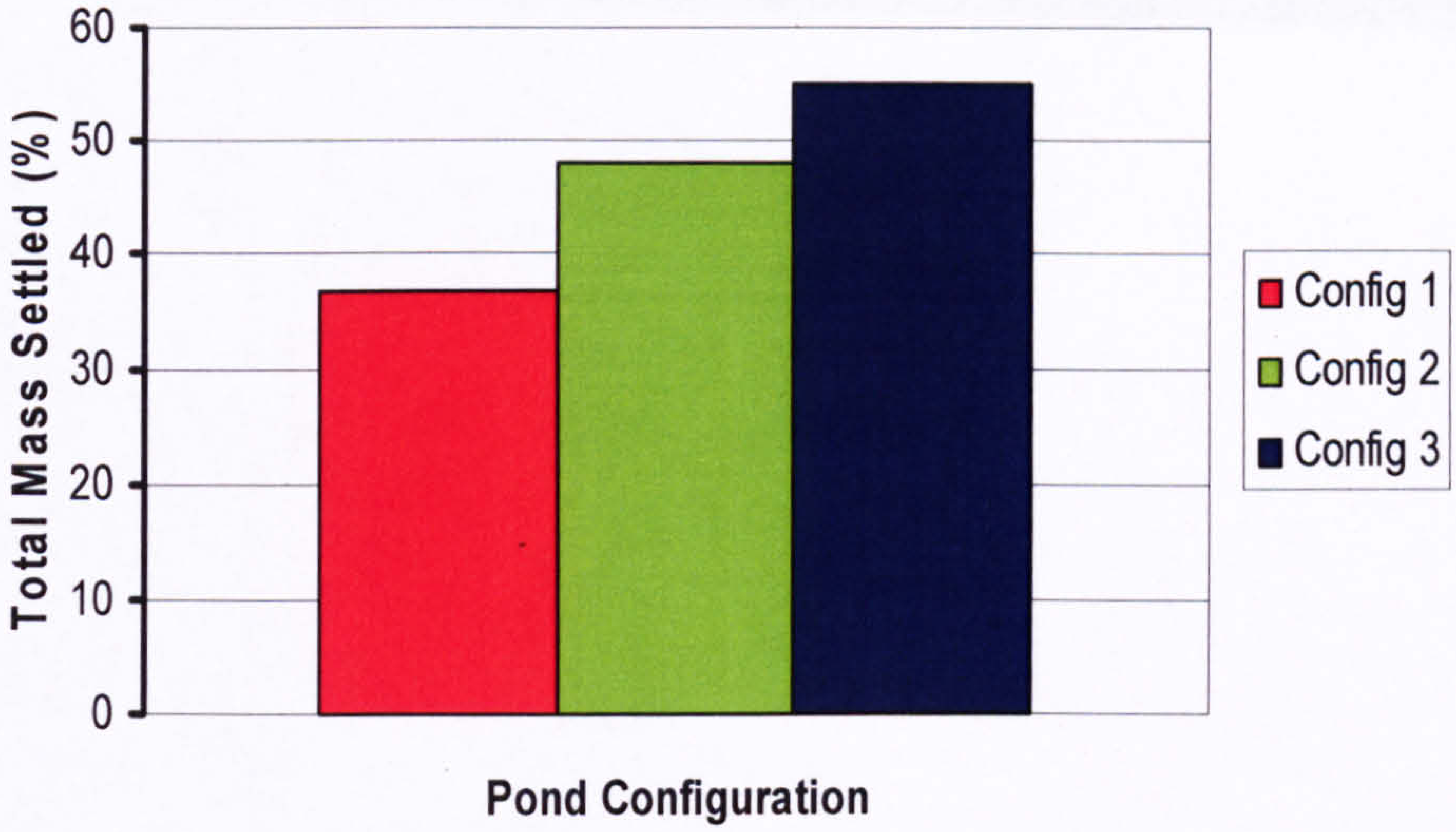


Figure 6.9: Water Quality performance of 3 pond configurations designed to meet water quality criteria for a small (Q90) storm

Since the basins have been sized to contain only the first 15mm of the storm event, the consequent reduction in pond radii (compared to that of the ponds in Section 6.3) results in all three ponds failing the flow attenuation criteria (shown in red font in Table 6.4) Configuration 3, the dual-level outlet pond performed best achieving a peak flow reduction of 35%, while Configuration 2, exhibited the worst flow attenuation performance reducing peak flow by only 2%. In terms of pond water quality performance, the mass of sediment removed was low in all configurations. Despite the ponds being able to capture the treatment volume (the runoff associated with 15mm of runoff), they were too small to contain the whole storm runoff volume. This resulted in flow over the weir in all three cases, which reduced retention times, and therefore sediment capture, in the ponds. As demonstrated in the sensitivity analysis in Chapter 5, the occurrence of weir flow is detrimental for water quality performance. For the 1 in 2 year storm, Configuration 3 had the longest retention time of 2.35 hours and removed just over 29% of incoming sediment, the greatest mass removal of all three configurations. The permanent pool volume in each pond was the same, therefore the improved performance of Configuration 3 can be attributed to the availability of both TSV (enabling storage of stormwater and lengthening the filling time before the weir is required to operate) and the increased pond surface area (as opposed to increasing the depth) as shown in Figure 5.8 in Chapter 5. All three ponds designed using this approach failed the sediment removal criteria of 80%.

In the Q90 event only pond configuration 3 reduced peak outflow by more than 50% of the peak inflow, thus achieving the flow attenuation target (Table 6.4). Sediment removal was in the range of 36-55% for all three configurations, with the dual-level outlet pond, Configuration 3 again achieving the greatest removal of sediment mass and having the longest residence time, as in the previous simulation (for the 1 in 2 event). This is in keeping with simulations undertaken in Chapter 5 which indicated that a permanent pool volume with a large surface area performs particularly well in capturing sediment. As with simulations for the 1 in 2 year event, all pond configurations designed to meet water quality guidelines failed the 80% sediment removal target.

As with simulations conducted in section 6.3, performance for both flow attenuation and water quality was better for the smaller storm magnitude under this design approach, in which the pond was sized to retain the treatment volume, irrespective of storm size. In general, both flow attenuation and water quality performance is poor for all configurations using this design approach - with all configurations failing the flow criteria for large storm events and with sediment retention for the daily Q90 event remaining below 55% for all configurations. In these simulations where the design approach focused on water quality, Configuration 3 performed best out of all the pond configurations for both flow and water quality, due to the availability of TSV for the storage of incoming stormwater and the increased surface area.

6.4.3 Summary and Discussion

Table 6.5 summarises the sediment attenuation results presented in Section 6.3.3 and Section 6.4.3 for ponds designed to meet flow attenuation and water quality treatment respectively. It clearly shows that when ponds are designed to meet the flow criteria set out in Chapter 4, all ponds meet those criteria, and water quality performance ranges from 47-98% but, as previously stated, in all cases performance is better for the smallest storm size. When ponds are designed to meet the new suggested UK water quality standards, the sizing of ponds to only contain one Vt has a negative impact on both the flow and sediment attenuation performance, with sediment capture never being greater than 55%. Results from these simulations suggest that the suggested change in design guidelines for water quality ponds from 4Vt to 1Vt is perhaps too large a decrease to provide adequate water quality benefits.

If ponds were required to capture 80% of sediment, which is typically the threshold used in many design manuals (Table 6.2), from the simulations conducted here, the only pond that would meet the criteria is Configuration 2 in the Q90 event designed to meet flow attenuation criteria. However, the good performance of this pond is entirely related to the size of its permanent pool (and corresponding surface area), which is an order of magnitude greater than any other pool investigated. Consequently, whilst this pond shows good performance it does not represent an optimal design. With the data available for Linburn pond, this issue of optimal pond design using different outlet designs, in comparison to land take required to meet a particular performance target, is investigated in Section 6.6.

Table 6.4 Permanent pool volumes and the percentage total mass of sediment settled (TMS) for three pond configurations in 1 in 2 year and Q90 storms in ponds sized for flow attenuation and water quality treatment. Data highlighted in red indicate cases which failed flow attenuation criteria.

	Pond Configuration		
	Configuration 1	**Configuration 2**	**Configuration 3**
Designed for Flow			
Permanent pool volume (m^3)	0	53014	6175
TMS for 1 in 2 year event (%)	47	78	50
TMS for Q90 event (%)	52	98	70
Designed for Water Quality			
Permanent pool volume (m^3)	0	2550	2550
TMS for 1 in 2 year event (%)	23	21	29
TMS for Q90 event (%)	37	48	55

6.5 The effect of multiple storms on pond performance

According to suggested UK design standards for both flow attenuation and water quality enhancement, a pond is sized based on approaches that use a single design storm. However as discussed in Chapter 4, in temperate climates, such as in the UK, storms may occur in quick succession often with very short antecedent periods in between. This poses a challenge for pond performance, since available pond storage volume is not only dependant upon the design storage capacity, but on a pond's ability to drain down in the inter-storm period, to provide storage for subsequent events. When designed to meet water quality standards, ponds are sized to contain Vt beneath the lowest outlet. This approach takes no account of probable storm size, and thus the retention of a large or a secondary inflow event may not be possible under such a design.

Furthermore, the physical processes that govern the build-up and wash off of urban sediments have an important role in the water quality of ponds. It is assumed that sediment build-up on a surface will increase exponentially up until a steady state when the rate of accumulation is equalled by the rate of loss from wind or traffic induced currents [*Sartor and Boyd*, 1972]. On the basis of this assumption, then it can be further assumed that shorter antecedent periods between storms will provide less time for sediment to accumulate on surfaces. It is well recognised that small, frequent storms are most significant in terms of impact on water quality. This section examines the impact of multiple small storms (which will undoubtedly occur regularly with short antecedent periods) on sediment capture behaviour and water quality improvement in the three pond configurations used previously in this chapter.

6.5.1 Pond design and the effect of multiple storms

Chapter 4 investigated the unknown effects of multiple storms on pond flow attenuation particularly under changing climatic conditions. Correspondingly little is known about how multiple storms affect sediment retention in ponds designed to meet flow attenuation standards (i.e. sized to attenuate the design storm flow peak by 50%) or water quality standards (i.e. sized to contain a permanent pool volume equal to the treatment volume, Vt).

To investigate these questions, different scenarios of small storm events were routed through the three pond configurations introduced in Section 6.3, sized to meet water quality standards (whereby the pond is sized to contain a permanent pool volume equal to Vt). The storm used was the Q90 event, since small frequent events are thought to be most critical in terms of water quality management. Two different scenarios were investigated, (1) two storms with one flush of sediment entering the pond with the first storm inflow and with no antecedent period, (2) two storms with two flushes of sediment entering the pond, one with each storm and with no antecedent period. The results from these simulations were compared to the base case of one storm with one flush of sediment.

Figure 6.10 shows the output from a storm inflow with two Q90 events occurring simultaneously with no antecedent period and two equal sediment inputs, one with each storm in Configuration 2. This scenario would only be physically realistic, if sediment were being remobilised in the catchment having not reached the pond during the first event. This represents a 'worst case scenario' for a pond. Note that for this storm size, sediment is washed in for the duration of the whole storm event. There is a clear reduction in sediment concentration between the inflow and outflow, illustrating the role of the large permanent pool in Configuration 2 in diluting incoming sediment concentration.

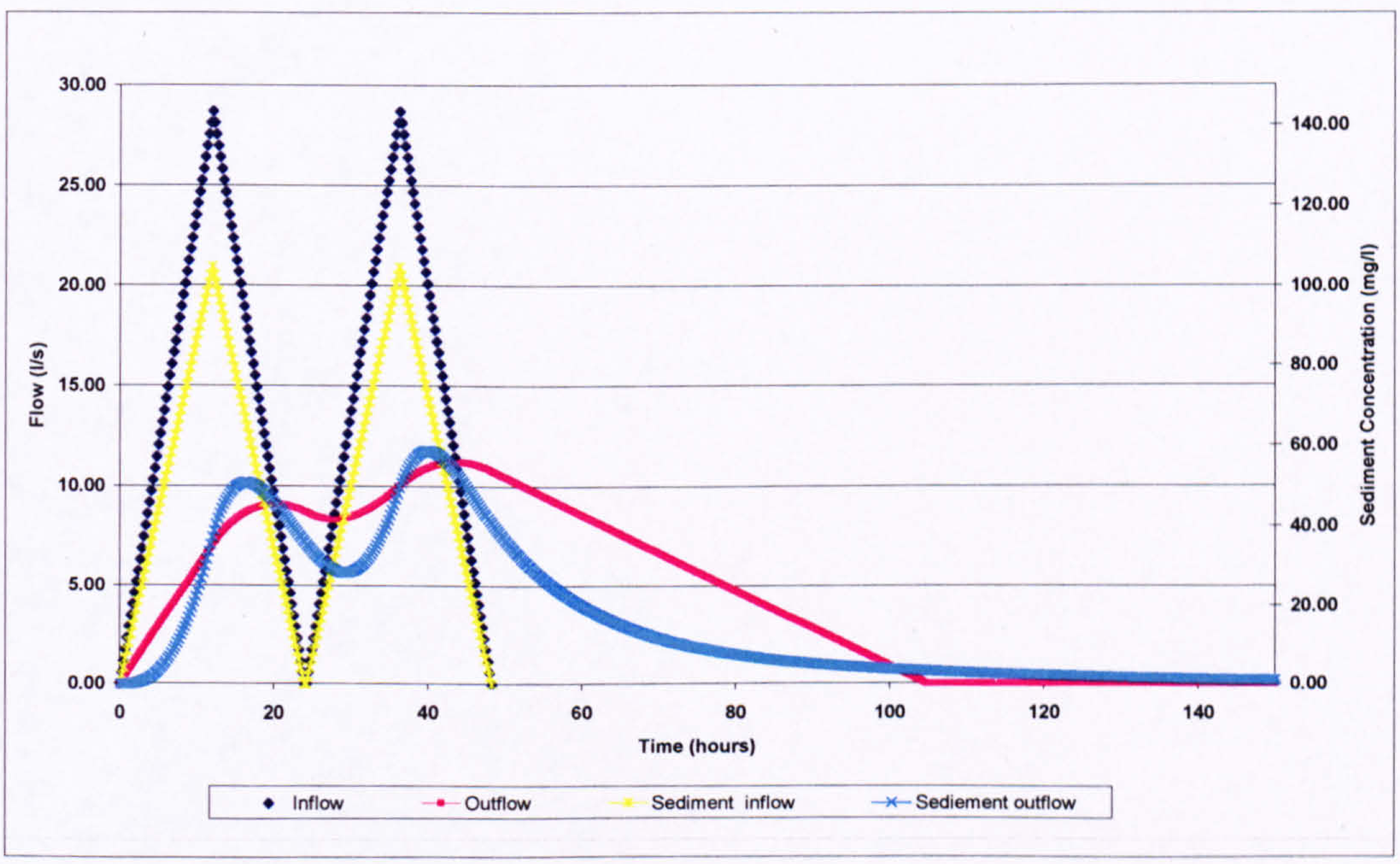


Figure 6.10: Pond inflow, pond outflow and sediment inflow and outflow for pond configuration 2 with an inflow of 2 Q90 storm events with 2 sediment inflows and no antecedent period.

6.5.2 Summary of results for multiple events using three different pond configurations tested against both flow and water quality criteria for initial pond sizing

Figures 6.11 and 6.12 show pond water quality performance for three storm event scenarios in each of the three pond configurations sized to meet flow attenuation criteria and water quality treatment standards.

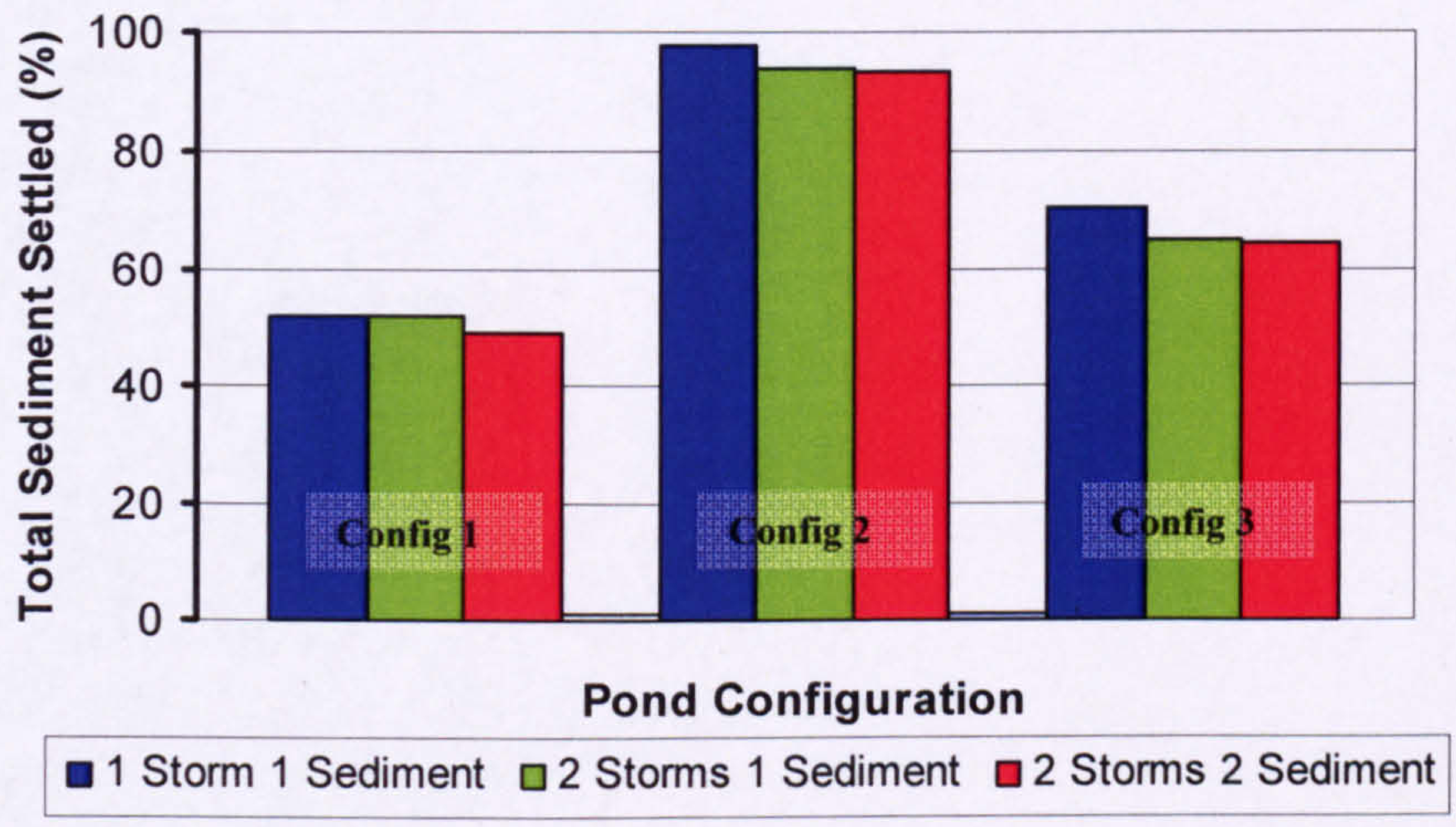


Figure 6.11: Total sediment settled in three pond configurations designed to meet flow design criteria, for various small storm sequences: 1 storm with 1 sediment inflow, 2 storms with 1 sediment inflow and 2 storms with 2 sediment flushes respectively.

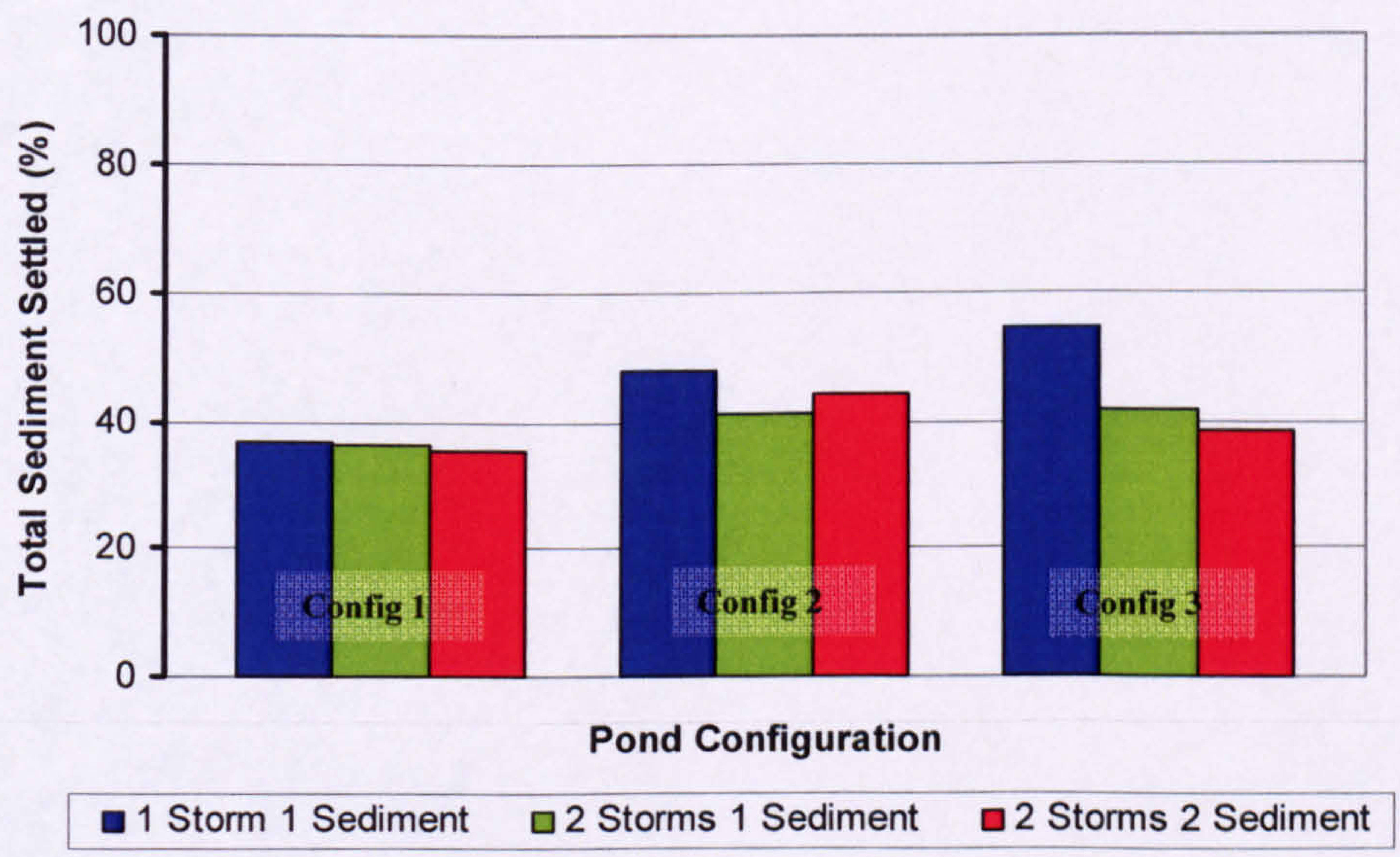


Figure 6.12: Total sediment settled in three pond configurations designed to water criteria, for various small storm sequences: 1 storm with 1 sediment inflow, 2 storms with 1 sediment inflow and 2 storms with 2 sediment flushes respectively.

Figures 6.11 and 6.12 clearly show that the impact of two sequential storms on the total sediment settling in the ponds is not significant when the ponds are designed to meet either the flow or water quality standards; the individual performance of each configuration appears not to be greatly affected by changes in storm sequence and sediment inflow conditions.

However, the decline in water quality performance on shifting the design emphasis from flow to water quality is clearly illustrated by these results. Taking the 2 Storm, 1 Sediment Flush scenario as an example, the overall decline in sediment retention on changing emphasis from flow to water quality design was least for Configuration 1 with a difference of 16%, slightly poorer for Configuration 3 (23%) and worst for Configuration 3 (53%). The results also suggest that multiple storm and sediment inflow sequences do not have a significant effect on the water quality performance of ponds whether designed to meet flow or water quality standards.

These results contrast with the significant effect that multiple storm sequences had on flow attenuation performance in Chapter 4. Further observation of the flow hydrographs suggests that for Configurations 2 and 3, the expected flushing out of sediment by a second sequential storm did not occur for two reasons:

(1) The presence of a very large permanent pool (which enables dilution of sediment concentrations)
(2) The size of the storm volume in comparison to the pool volume (which is too small to create high enough flows to inhibit settling)

Since Configuration 1 is designed to have no permanent pool, the assumptions made in (1) and (2) above do not apply directly.

6.5.3 Infiltration basin design

Ponds in which multiple storm sequences are most likely to affect water quality performance, are those in which the storms are too large to be retained (i.e. inflow volume is larger than Vt). To investigate this, a fourth configuration was examined – the infiltration basin (Figure 6.13). The infiltration basin is essentially the same as a detention basin

(Configuration 1), however TSV is much smaller (the weir crest is at a lower elevation, 1.5m, and is the only outlet), since the primary mechanism of drainage in this kind of basin is infiltration to the soil. Since TSV is so small in this type of design, it represents a worst case scenario for a pond. However, it should be noted that the mechanism of infiltration is not modelled here.

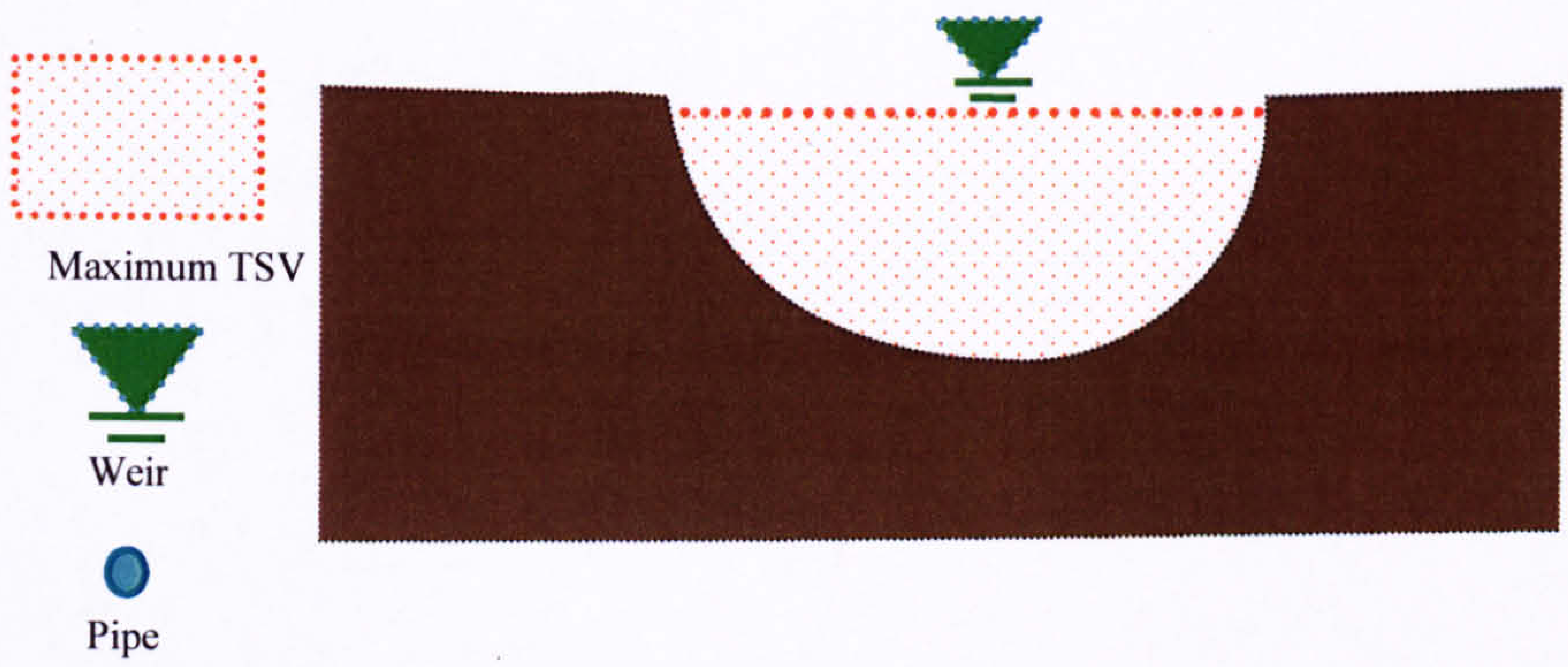


Figure 6.13: Pond Configuration 4 – an infiltration basin. Note the basin has no pipe since the primary mechanism of drainage is infiltration through the soil.

A number of simulations were conducted, to assess the effect of multiple storms on the water quality performance of an infiltration basin. The basin had the same dimensions as the detention basin, with a radius of 16.45m and surface area of 850m^2. The storm and sediment sequences that were chosen for the analysis are the same as those used in the simulations in Section 6.5.1 and 6.5.2, using a 24-hour duration Q90 event.

6.5.4 Results

Table 6.6 and Figures 6.14, 6.15 and 6.16 show the simulation results for an infiltration pond sized to contain Vt under the three different flow and sediment inflow scenarios. In Figure 6.14, the effect of a first storm on sediment settling can be seen. Initially as storm and sediment inflow begin, sediment settling (represented by the green line) begins to increase. However as the water level reaches the crest of the weir and flow over the weir (represented by the red outflow line) begins, the 'flushing through' effect is evident from the occurrence of sediment in the outflow, with little opportunity for settling. It is only once the peak of the outflow has passed that sediment settling resumes once more. This effect is observed in all storm and sediment sequences in Figures 6.15 and 6.16. The results for water quality performance of the infiltration basin show a greater effect of multiple events than in the other three configurations, with total mass of sediment settled decreasing from 53% for the single storm to 39% in the two storm one flush scenario (Table 6.6).

Table 6.6: Total sediment settling in an infiltration basin sized to contain Vt for different Q90 and sediment inflow scenarios

Scenario	Total mass settled (%)
1 storm 1 sediment flush	53
2 storms 1 sediment flush	39
2 storms 2 sediment flushes	42

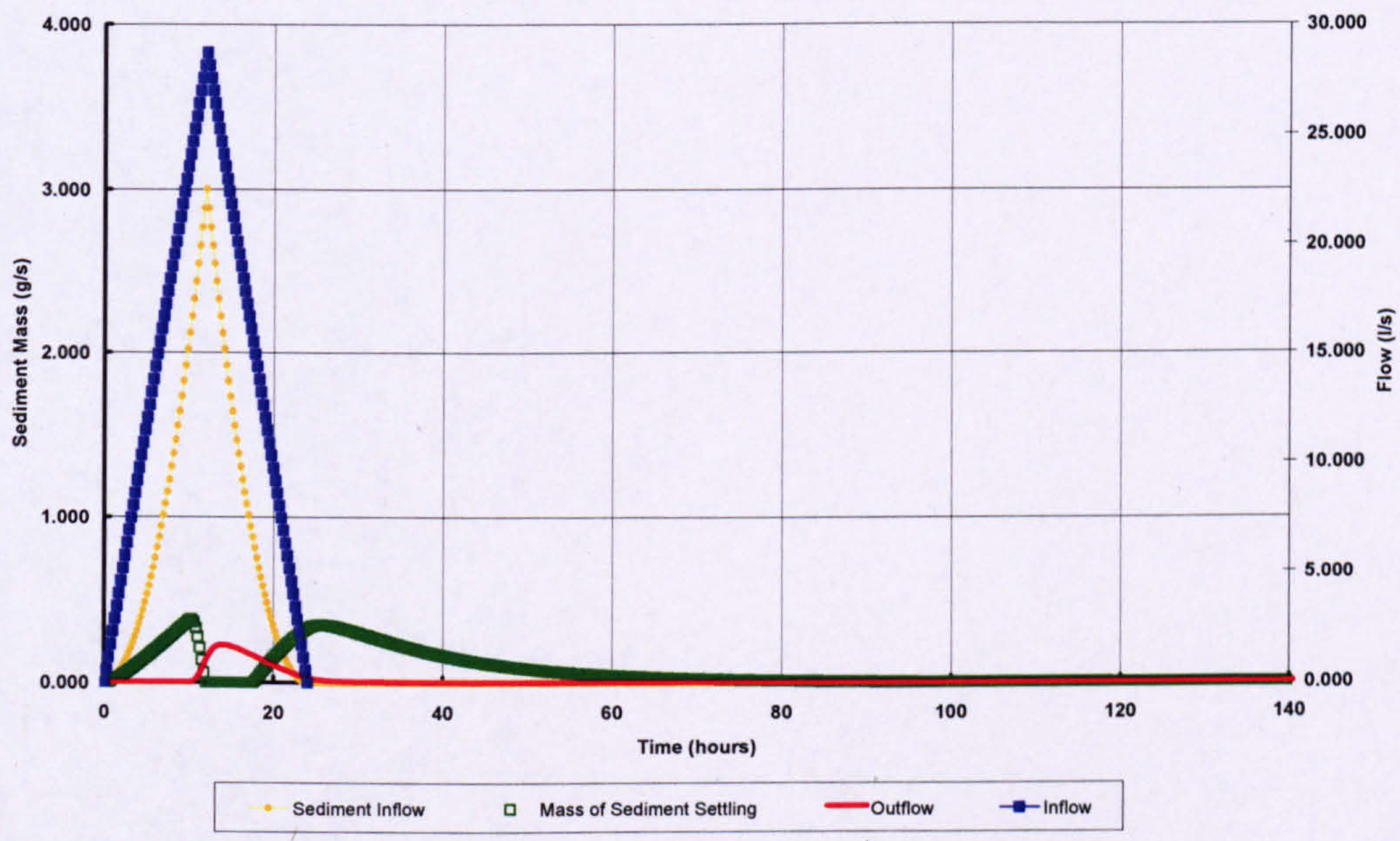


Figure 6.14: Sediment mass inflow, sediment mass settling, and outflow during a Q90 storm in an infiltration pond

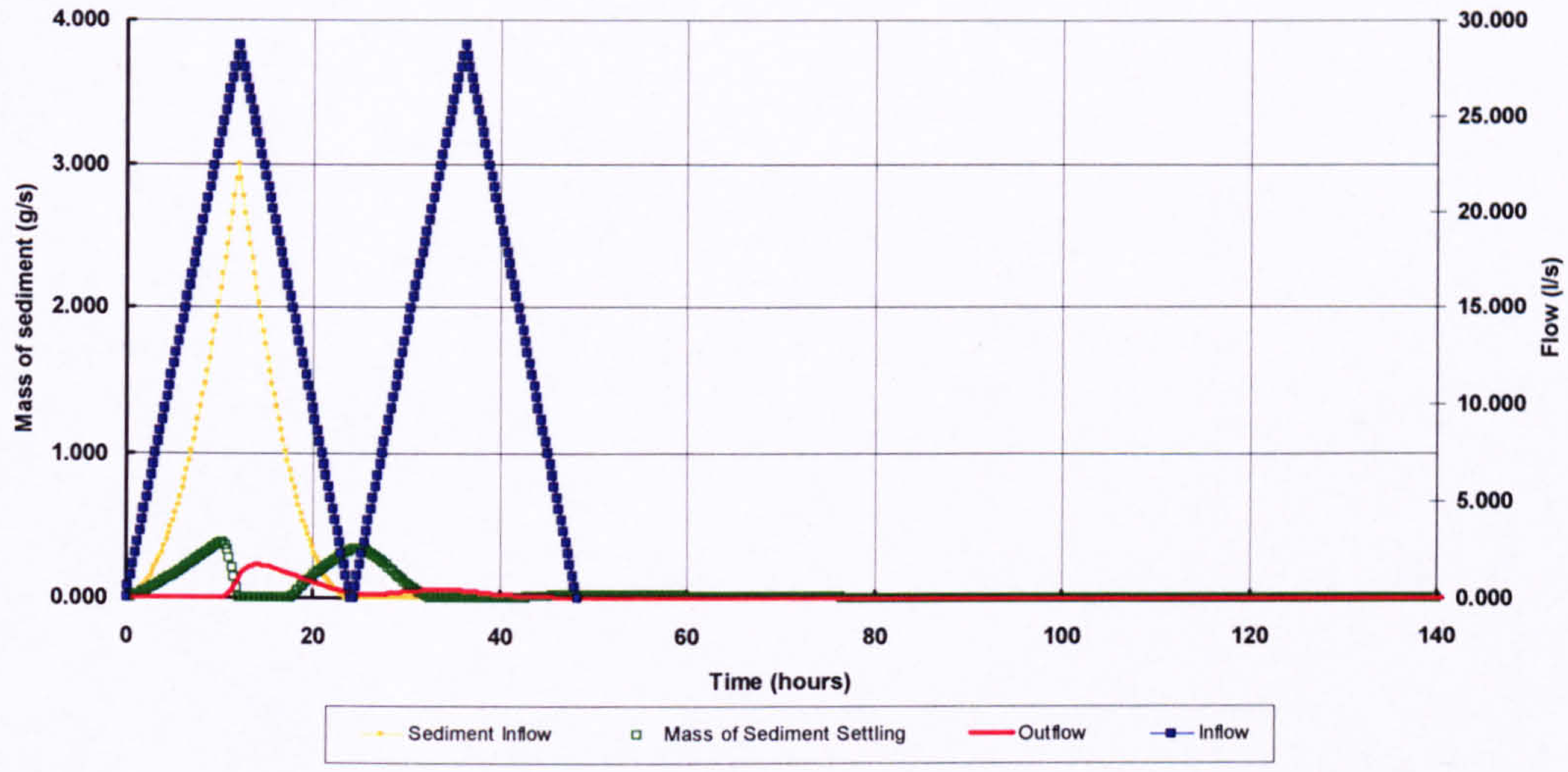


Figure 6.15: Sediment mass inflow, sediment mass settling, and pond outflow during two Q90 storms with sediment flush in an infiltration pond

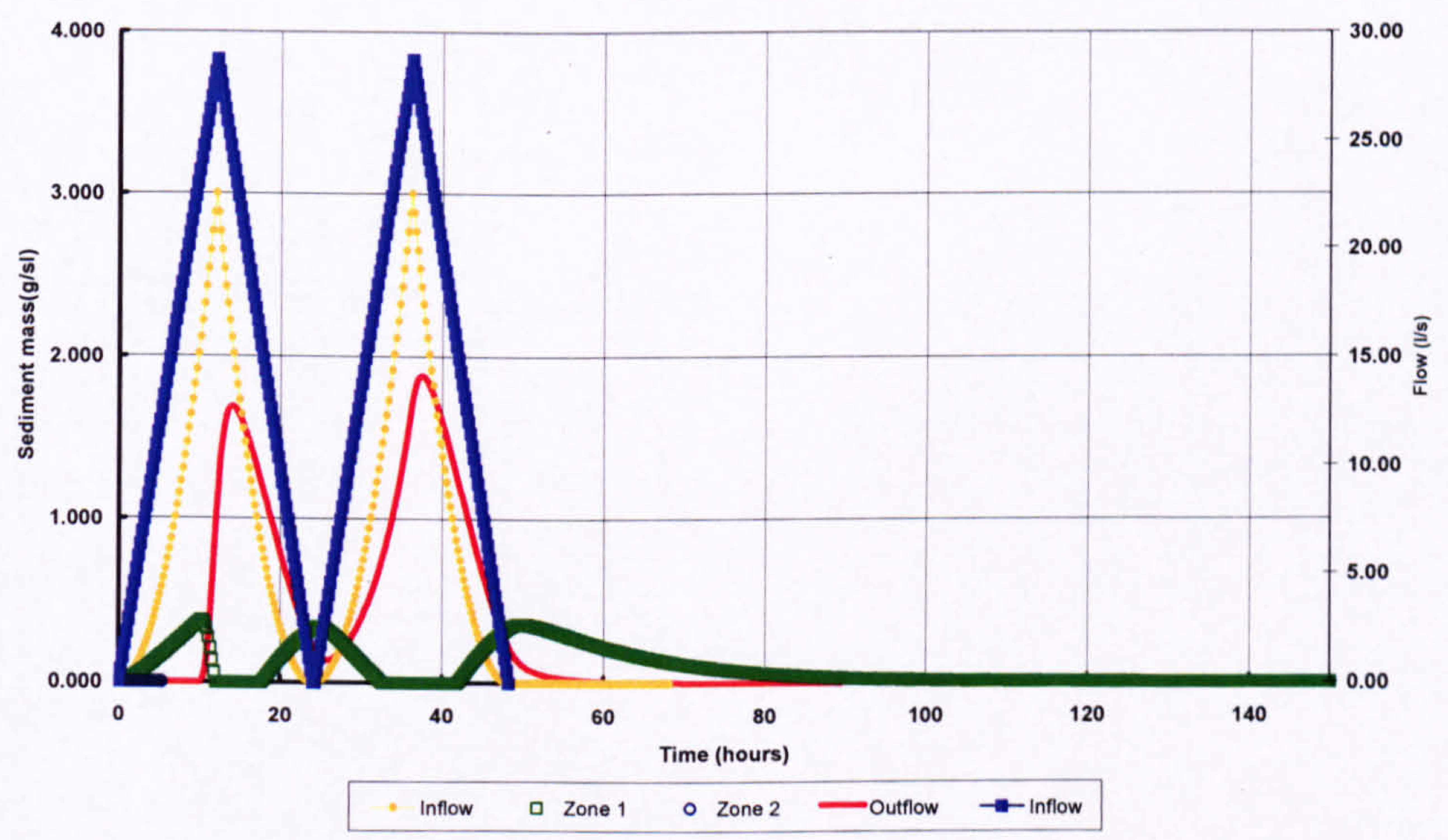


Figure 6.16: Sediment mass inflow, sediment mass settling and pond outflow during 2 Q90 storms with sediment flush in an infiltration pond

These results demonstrate that under these particular conditions, the effect of multiple storms on pond water quality performance can be observed, in contrast to earlier simulations where the flushing of sediment through the pond was not as apparent. The importance of considering two storms for sediment attenuation performance, however, is clearly relatively minor in comparison to that found for flow attenuation in Chapter 4.

The infiltration basin configuration was selected as an extreme case, to demonstrate that multiple storms can have an effect on stormwater basins and to verify that the pond model could reproduce sediment flushing behaviour under these conditions. It is unlikely that such a basin would be a feasible option for stormwater management in Scotland, since for this type of basin to have the full Vt available at the start of a storm would require efficient and full drainage between storms. Such drainage is not consistent with the Scottish climate (both in terms of rainfall quantity and the distribution of antecedent periods between rainfall events) or with Scottish soils, which tend to have low infiltration rates. Figure 6.17 shows the frequency of antecedent period length in Scotland derived from BADC data. Eight stations from the West of Scotland (Barcaldine, Clachan, Glenfyne, Mull Grulline, Skipness, Ormsary, Islay Ellabus, and Garvie Farm) and eight stations from the East of Scotland (Braemar, Clatto, Prestonpans, Gullane, Glenogil, East Linton, Dunglass and Skedsbush) were used. Only stations with full records spanning the period 1961-1999 were used. Figure 6.17 shows that for over 45% of rainfall events, there is only 1 day for the pond to drain before the next rainfall event.

Whilst the design of infiltration ponds is not of interest in Scotland, there are drier regions of the UK where such basins are appropriate. These results highlight important design considerations for infiltration basins at such locations.

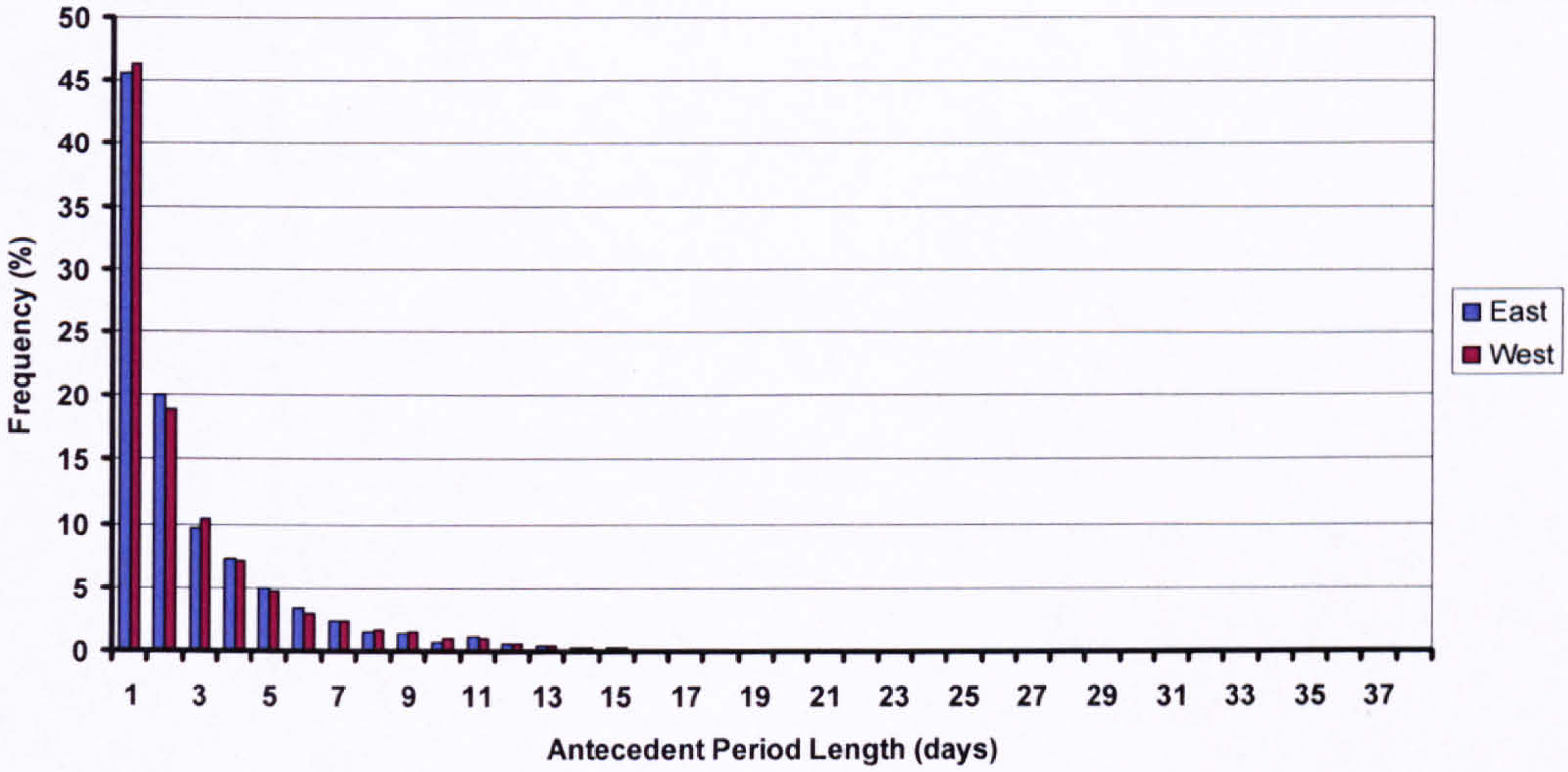


Figure 6.17: Percentage frequency of antecedent period length derived from 8 stations in the East and 8 stations in the West of Scotland (produced using data from BADC).

6.6 Water Quality Performance of Linburn Pond

Linburn pond, a retention pond currently in use at the Dunfermline Eastern Expansion Site in Fife, Scotland was introduced earlier in section 4.4 of Chapter 4. Its configuration is largely similar to that of pond Configuration 2, having only a single-level outlet (comprising 4 weirs) and thus having a water level that is always at or above the elevation of the weir crest. The following section investigates its current water quality performance. Later sections investigate how the re-design of Linburn pond to meet the flow attenuation criteria proposed in Chapter 4 affects the pond's water quality performance. The final section proposes an optimal design of Linburn pond that meets *both* the flow and water quality criteria.

6.6.1 Water quality performance of the current Linburn pond

Currently Linburn pond is designed with 4 single-level 90° v-notch weirs situated at 2.8m above the base of the pond. Because this is the only mechanism for pond drainage, the water level is always at the point of overflow at the weir crests and, therefore, there is never any temporary storage volume available in the pond (Figure 6.18). In Chapter 4 such a design was shown to exhibit extremely poor flow attenuation performance.

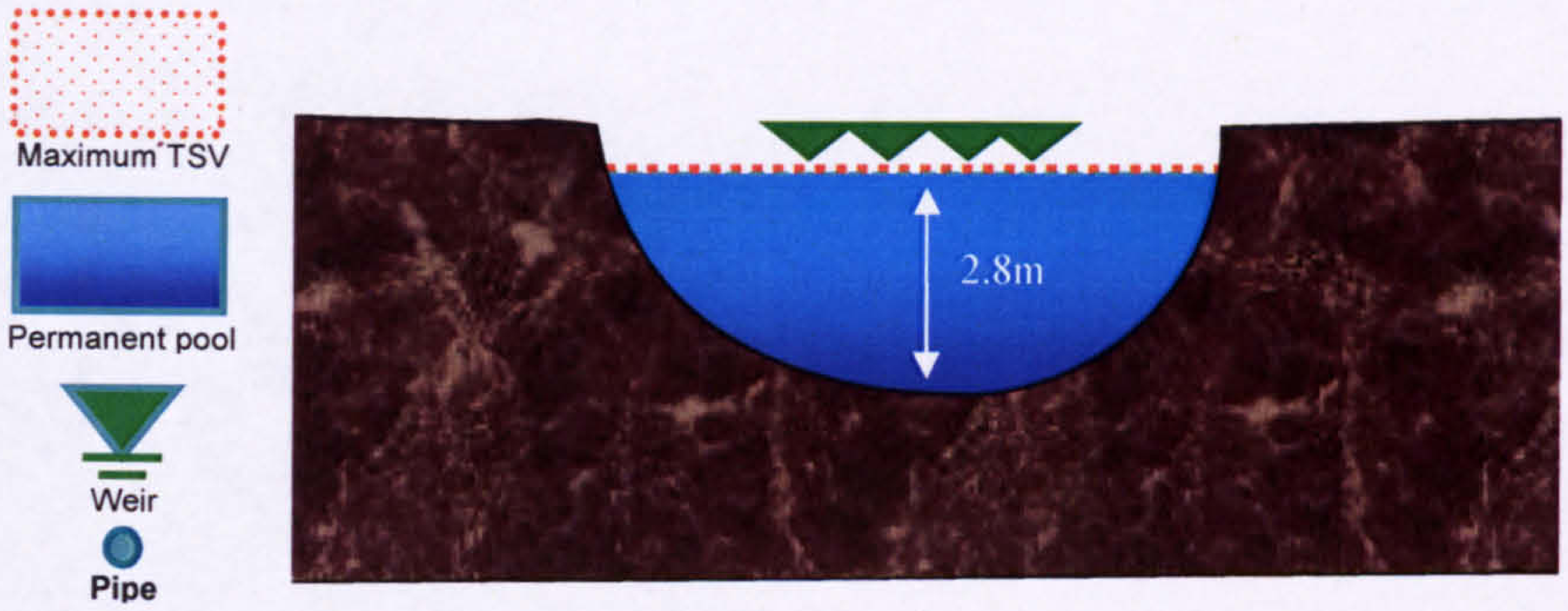


Figure 6.18: Schematic representation of Linburn pond. Note the presence of 4 single-level v-notch weirs, which without the presence of a secondary outlet ensure water levels are always at the weir crest.

Using the same triangular inflow hydrographs as in earlier sections, (the 1 in 25 year event, the 1 in 2 year event and the Q90 event of 24 hour duration with a single sediment influx) the water quality performance of Linburn pond was assessed by simulating the percentage of total sediment mass settling in the pond. Figure 6.19 shows results for the Q90 storm. As can be seen from the plot, sediment inflow occurs throughout the duration of the storm and there is a significant reduction in sediment concentration between the inflow and the outflow, which is a reflection of dilution by the large permanent pool.

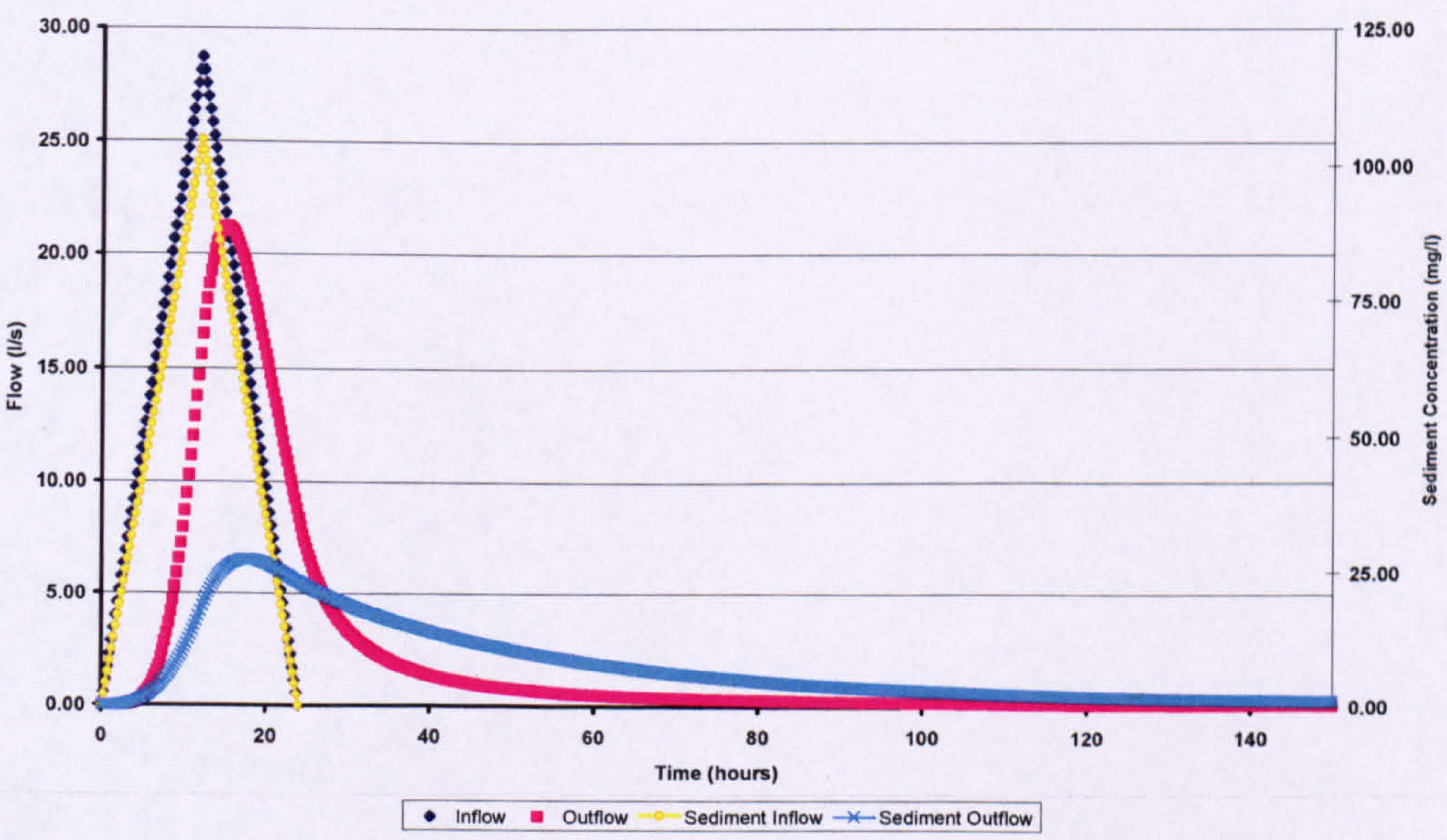


Figure 6.19: The Q90 event in Linburn pond (current design). Note that outflow occurs only through the 4 weirs which are the only outlets in this design.

Table 6.7 illustrates Linburn pond performance for both flow and sediment attenuation. As before, the flow attenuation performance was extremely poor, even for the Q90 event. For small, frequent storms as represented by the Q90, Linburn pond was able to remove 77% of the sediment mass. For storms of a larger magnitude, performance was much reduced removing 33% of the sediment for the 1 in 2 year event and 24% for the 1 in 25 year event. Under the design currently in place, Linburn Pond would fail to meet the 80% sediment removal target for all simulated storms.

Table 6.7: Performance of Linburn pond under the current design for three different storm magnitudes of 24 hr duration

Current Design	1 in 25	1 in 2	Q90
Flow			
Radius required to meet 50% (m)	40	40	40
Pond Surface Area (land take)	5026.55	5026.55	5026.55
Peak Flow Reduction (%)	4.0	7.0	26
Water Quality			
Total mass settled (%)	24	33	77

6.6.2 Linburn pond performance: An optimum design

Here a pond configuration is proposed to maximise the performance of *both* flow and water quality in the pond. The improved design assumes that since the pond is already well-established, the radius must be kept at 40m. The parameters adjusted were the weir crest elevation, (restricted to a maximum of 3m for health and safety reasons), the elevation of a secondary pipe (restricted to a minimum of 1m to sustain aquatic plant and animal communities and prevent bed scour) and the pipe diameter. It was found by successive adjustment of each parameter, that the optimum performance for both pond functions was a weir crest of 3m, a pipe elevation of 1m and a pipe diameter of 0.05m.

Table 6.8: Pond flow and water quality performance for 3 storm magnitudes under the current Linburn pond design and a new improved design

	1 in 25	1 in 2	Q90
Radius required to meet 50% (m)	40	40	40
Pond Surface Area (land take)	5026.55	5026.55	5026.55
Flow (Peak Flow Reduction)			
Current Design	4.0	7.0	26
Optimum Design (Chap 6)	95	96	91
Water Quality Total mass settled (%)			
Current Design	24	33	77
Optimum Design (Chap 6)	78	72	89

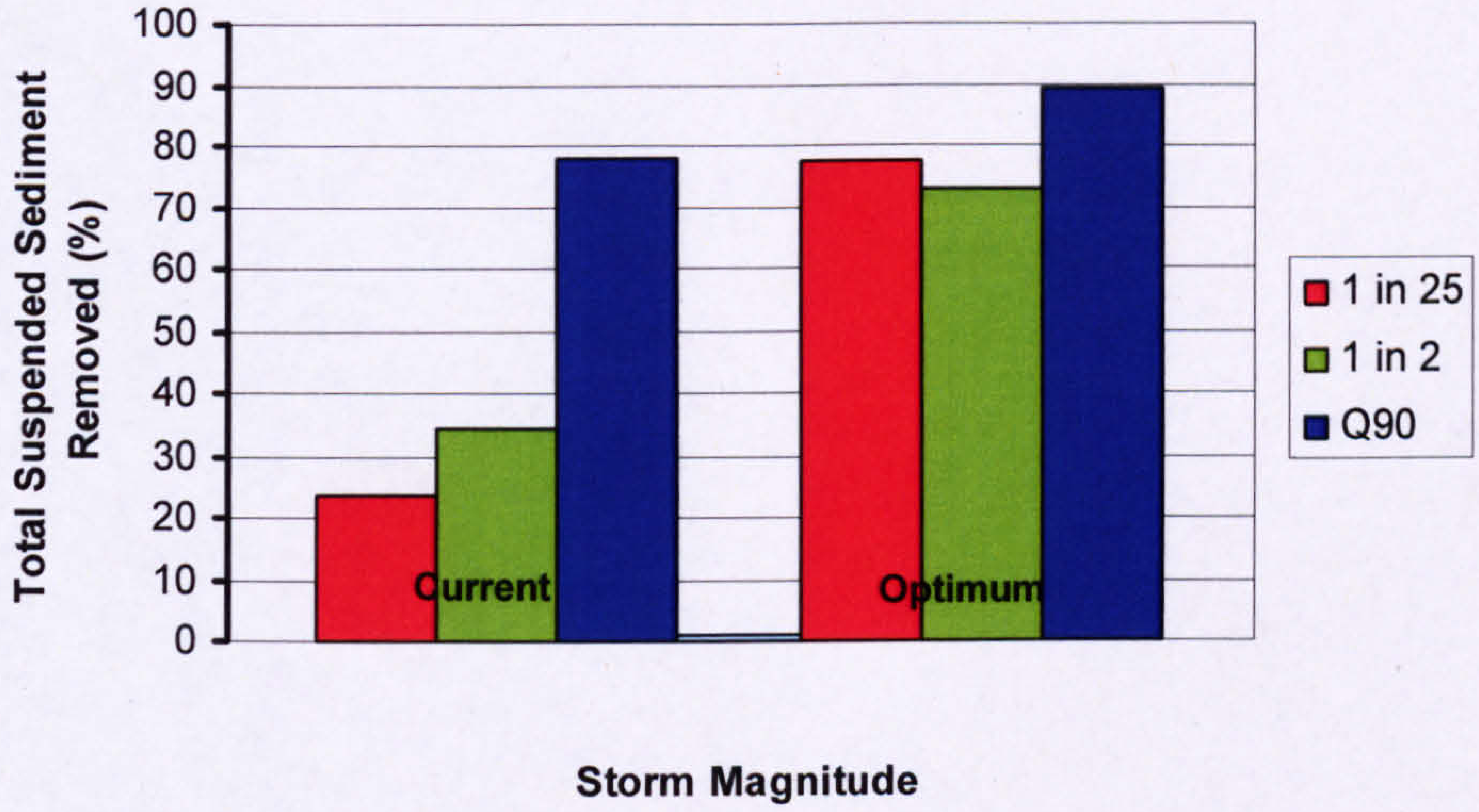


Figure 6.20: Water quality performance of Linburn pond under three design scenarios for 3 different storm magnitudes

Figure 6.20 and Table 6.8 show flow and water quality results for the current Linburn pond and the optimum design described above. These clearly show that the optimum pond far outperforms the current configuration for both flow and sediment attenuation, for all three rainfall events. The optimum pond design meets both the flow (50% reduction for all events) and the water quality (80% reduction for Q90 events) design criteria, even achieving an 80% sediment attenuation for the 1 in 2 year event. This represents a highly significant improvement in pond performance, which could be achieved for relatively little cost. In terms of the current water quality guidelines, this optimum performance has been achieved for a pond size of 1.4Vt.

The optimum pond was able to maximise the performance of both flow and sediment attenuation because the design approach considered each in turn. First, flow attenuation was ensured; sizing the pond to meet the specified flow criteria. Once this had been achieved using a dual outlet configuration, the permanent pool volume was gradually increased until the pond met the desired water quality targets – hence providing both for flow and water quality requirements

6.7 Summary and Discussion

In the past it has been commonplace for stormwater ponds to be designed with flow attenuation as the primary focus. Pond design that focuses on water quality enhancement or that which considers both water quality and flow issues is a relatively new concept in the UK [*Campbell et al.*, 2004]. Previously, in designing retention ponds to meet water quality standards, the approach recommended in the UK sized the pond to contain 4Vt. More recently a treatment volume of just 1 Vt has been suggested [*McLean et al.*, 2005]. Results from simulations in this chapter have demonstrated that performance is inadequate for both flow and water quality when ponds are sized to contain a permanent pool volume of only 1 Vt. However, with a dual outlet configuration excellent performance in terms of both flow and water quality were achieved by a pond of volume 1.4Vt.

It is clear from the simulation results that the water quality performance of retention ponds is in part related to pond flow performance. In numerous cases, the total mass of sediment settled was shown to be dependant upon the flow through the outlet and thus on the loss of sediment in the outflow. This means that the design of the outlet device is critical not only for flow control and minimising land take, as shown in Chapter 4, but also for sediment capture in a pond.

The simulations undertaken in this chapter have also enabled the quantification of potential sediment removal in *detention* basins. As far as can be determined, this is the first time that such findings have been presented for detention basins in the U.K. The results indicate that detention basins can potentially retain a high proportion of sediment particles. In the simulations undertaken in this chapter, sediment removal ranged from 23% - 52%. This is significant sediment removal considering that these basins are designed only to attenuate flows and are devoid of a permanent pool (the main mechanism reported in the literature responsible for sediment settling in ponds), having only a temporary pool of water at any given time during a storm.

In Chapter 4, simulations were conducted to investigate the effect of climate change on pond flow attenuation performance. The results showed that larger magnitude storm events and shorter antecedent periods would adversely affect flow performance. The effects of climate change on pond water quality were not investigated here (Chapter 6) per se, however the decline in performance that occurs under both flow and water quality design with increasing storm magnitude is clearly demonstrated. In order to operate with maximum TSV, ponds must drain down in the short inter-event period, demonstrated to be 1-3 days in Section 6.5.4. This requires careful selection of adequate pipe diameters in order to restrict the outflow and hence provide long filling up periods for smaller events whilst reducing the risk of overflow (and loss of sediment in outflow) for larger storm magnitudes.

There are currently no available predictions of how climate change will affect the build up and transport of sediment in the built environment. Processes that affect the availability of sediments such as deforestation and soil erosion might be exacerbated by climate change, however, such processes will probably be concentrated in rural or developing regions rather than established urban areas. Other possible long-term trends that might affect sediment accumulation such as increased fuel consumption and vehicle emissions are likely to be concentrated in the urban environment. There are currently no data available to make informed assumptions on changes in sediment particulate loads and how they will affect pond water quality performance. There has, however, been much research into how climate change might affect the hydrological regime. If General Circulation Model predictions are correct, then it is possible that the UK might experience an increase in hydrological extremes. Therefore, based on the results presented in this thesis, it can be assumed that if such conditions exceed pond design criteria, the flow attenuation and water quality performance of storm water ponds will be adversely affected.

In Chapter 2 it was suggested that flow and water quality are competing issues in stormwater pond design. However, results presented here suggest that designing for flow attenuation is actually beneficial for water quality performance, since it provides large permanent pools and with sufficient temporary storage, weir operation is restricted to only the most extreme events, limiting the loss of sediment in the outflow. The approach used in this chapter to successfully maximise performance for both flow and water quality designed a pond primarily to meet flow attenuation standards before increasing the permanent pool volume to meet water quality targets.

7 Summary of Results, Conclusions and Design Recommendations

This section of the work details the findings and conclusions of the thesis, however in brief, there are three main outcomes of this work. These are as follows:

- This is the first research of this kind to look at the effect of multiple storms on SUD systems, and results clearly demonstrate the inadequacy of current design guidance in only considering a single storm in the design process.
- This is also the first work to investigate the method of Vt for sizing water quality basins in the UK. Results in this thesis illustrate that this method is not adequate in designing ponds for high standards of water quality treatment
- Finally, the thesis presents a new, alternative method for designing retention ponds to provide the dual functions of flow attenuation and water quality enhancement. The procedure is simple to follow and enables very high standards of both functions to be achieved. It encourages careful design of the outlet device, recommending the use of a multi-level outlet and promotes the provision of an area for temporary storage of stormwater.

Retention ponds are designed to perform the dual purpose of attenuating high flows and improving the quality of water discharged into receiving streams and rivers. In this thesis, a new model of retention pond performance has been presented and validated. The model was applied to investigate both flow and water quality performance under single storm events, multiple storm events and climate change scenarios.

Simulations using the new pond model (Chapter 4) highlighted a number of inadequacies in the design criteria for retention ponds. Current design guidance in the UK is largely responsible for this since it encourages the design of ponds with single-level outlet devices which do not allow drainage of stormwater between storms and provide no temporary storage for the attenuation of storm inflow. Furthermore, guidance promotes the use of single event design criteria in determining the 'design storm'. Model results demonstrated this to be insufficient for providing good flow attenuation of multiple storm events - even when they are of a smaller magnitude than the design storm. The simulations presented here highlight the need to consider multi-event probabilities in retention pond design, however, such data is not currently available from UK rainfall data. Nevertheless, determining the probability of occurrence of multiple design storms and the time period between multiple storms would reduce the risk of pond failure during sequences of high inflow events.

Chapter 4 also investigated the effect of outlet device on pond performance. Improved flow attenuation was achieved when ponds were designed to have dual-level outlets, comprising a high level weir and a submerged pipe. Selecting the correct pipe size with respect to the storm magnitude is a key design consideration. Since both large, infrequent storms and smaller more frequent events occur in a typical hydrological year, selecting a pipe size to attenuate both to meet the pre-defined design standard is crucial in providing good overall flow attenuation performance.

The current design of Linburn Pond at DEX (the flagship site of Scottish SUDS design) is a typical product of the current UK retention pond design guidance. The pond has 4 single-level v-notch weirs that are set high in the pond. Simulations in Chapter 4 showed that this design does not provide any Temporary Storage Volume (TSV), prevents the pond draining after a storm and results in a pond that has immediate outflow with the onset of inflow. Simulations of this configuration showed the current design achieves very poor performance, failing to meet a realistic flow attenuation criterion (whereby peak outflow should be reduced at least to 50% of the peak inflow) even for the relatively small events simulated. This type of performance is consistent with the pond having no TSV. Application of the pond model to design improvements for Linburn Pond demonstrated that by including a dual-level outlet into the pond and by incorporating an impermeable pond liner to prevent groundwater inflow, performance could be improved significantly. The lined pond failed only twice throughout a typical year, while the unlined pond failed 18 times. Simulations also showed that when the storm magnitude was increased to reflect the effect of climate change, the pond radius would have to be increased by 7m to meet the flow attenuation criteria.

In Chapters 5 and 6 pond water quality performance was investigated using a new model of sediment capture. Much of the literature suggests a strong relationship between pond retention time and sediment settling. A sensitivity analysis in Chapter 5 showed that whilst there is a link there are several other factors that effect sediment retention. Most crucially, all simulations demonstrated that sediment capture was highest in those ponds with large permanent pools, primarily due to the effects of dilution. In all cases, the total mass of sediment settled in the pond was also dependent on the flow rate through the outlet(s) and hence on the outlet design. The operation of the weir (as opposed to the submerged pipe alone) increased the mass of sediment in the outflow in every case, adversely affecting the water quality performance of the pond. The design of the outlet device, therefore, is critical not only in terms of flow control, as shown in Chapter 4, but also in terms of the sediment capture in a pond. This is particularly true of the finer sediment fractions which are most likely to be lost through the outlet due to their tendency not to settle, which is exacerbated under the conditions created when the weir is operating. It is these fine particle sizes that are typically most associated with adsorbed pollutants such as heavy metals, typically found in urban runoff.

The currently favoured approach of designing ponds for water quality is based on a method that aims to contain only a single treatment volume (Vt). Investigations in Chapter 6 showed that ponds designed in this way failed both realistic flow and water quality standards since they were unable to contain and attenuate storm flows. Also their water quality performance deteriorated as storm size (relative to Vt) increased.

Three different pond configurations were investigated for their contribution to water quality; a detention basin, a single-level outlet retention pond, and a dual-level outlet retention pond. Performance across all simulations was best when the pond was designed to meet flow rather than water quality design standards. In all cases it was shown that when the pond was designed on the basis of one Vt rather than to attenuate flows by 50%, water quality performance declined. In most cases this was attributed to the lack of TSV, requiring the weir to operate to drain the pond, which resulted in higher loss of sediment through the weir, and the higher sediment concentrations that resulted from smaller permanent pool volumes. A fourth configuration, an infiltration pond, was introduced to demonstrate more clearly the effect of multiple storms on water quality performance. A clearer 'flushing' of sediment through and out of the pond was observed, in contrast to other simulations, when flow through the weir began and a clear deterioration in sediment capture occurred with multiple storms in the pond.

The water quality performance of Linburn Pond was also analysed in Chapter 6 and was shown to be reasonable for small, daily events but to be very poor for larger storms under its current design. An optimum pond design was achieved by incorporating a dual-level outlet, comprising a high-level weir and a lower-level small orifice thereby increasing the TSV by maximising the difference in elevation between the two outlets. This design achieved excellent sediment removal for all storm sizes.

7.1 Conclusions

Based on the results and analysis presented, the following conclusions may be drawn;

- Pond design is essential in determining pond performance. Pond flow attenuation was quantified, ranging from 0.2% reduction in peak flows in poorly designed ponds to a maximum of 97% achieved in a pond with a very large surface area. The optimal pond design (high performance with minimal land take) achieved a peak flow reduction of 96% when applied to the case study, Linburn pond.
- An investigation of pond flow attenuation performance indicated that a design that encourages the use of a multi-level outlet and considers the provision of an area of temporary storage (TSV) above the highest outlet is crucial in good pond performance. These design parameters exercise a critical control over pond flow attenuation.
- Good flow attenuation is dependent upon the complex operating cycle of the pond, whereby TSV contains and attenuates stormwater inflow and the outlet device(s) drain the pond. The combined effect of providing sufficient storage and draining the pond slowly enough to provide quiescent conditions for water quality treatment, yet rapidly enough to provide TSV for subsequent storms, is essential in ensuring the pond meets flow attenuation targets.
- Possible changes in future climate must be considered at the design stage. Ponds that are designed to only meet the design storm, with no extra storage provided for increases in storm magnitude (or for reductions in volume due to sediment accumulation) were demonstrated to be ill-equipped to meet design criteria, with frequent failure in flow attenuation. In the UK, ponds are currently designed in this way, without any consideration of possible increases in event magnitude. The

current guidance for Scotland does not account for losses in lined ponds due to the effects of evaporation, since these are thought to be negligible. However in other areas of the UK, a series of high temperature days could reduce permanent pool volumes and adversely affect pond water quality performance.

- The water quality performance of retention ponds was quantified, ranging from 21% total mass settled in a poorly designed pond to 98% in a pond with a very large surface area. The optimum pond water quality performance (optimum water quality performance with smallest land take) was 78%.
- Optimum water quality performance is dependant upon large permanent pools that dilute sediment concentrations and provide large residence times for sediment settling. However, TSV is also important in water quality performance. If there is not sufficient TSV, overflow through the weir increases the loss of sediment in the outflow and reduces the sediment capture of the pond.
- Designs that size a pond for sediment attenuation by capturing a single 'treatment volume' are inadequate in providing good water quality performance. Inevitably the ponds are too small to contain and detain even relatively frequent flow events, such as the Q90, which is likely to mobilise significant sediment in the catchment area. The design criteria are based on a calculation of the first flush event, which is associated with the first 12-15mm of runoff in the catchment. A review of this method for sizing ponds for water quality is essential if future pond design for water quality is to be improved.
- Retention ponds can be designed to achieve high standards of both flow attenuation and water quality enhancement. This can only be achieved by first designing the pond to pass the flow attenuation design standard (incorporating a climate change factor) and increasing the pond surface area to accommodate a suitable permanent pool volume for water quality. This method disregards the current UK water quality design guidance of sizing the pond to according to Vt. This alternative method is presented in full in Section 7.2
- Both retention ponds and detention basins can achieve high flow and water quality standards if designed well. The role of detention basins in sediment capture was quantified for the first time in this work and showed that they can play a significant role in water quality protection –without the need for a permanent pool.

- Current design guidance is inadequate for retention ponds in Scotland since it does not promote design that incorporates TSV, as illustrated by the Linburn Pond case study. Design guidance that is more site specific and considers flow attenuation, water quality, multiple events, groundwater interaction and the likelihood of future climatic variability will enable the design of ponds that meet current and future needs. New design guidelines to meet these needs are proposed in Section 7.2.
- As well as the poor design of the outlet, a failure to recognise the importance of groundwater in the Linburn catchment is largely responsible for the failure of Linburn Pond to attenuate flows. Design guidance that considers the role of groundwater in stormwater management is required to prevent further inadequate pond designs (such as Linburn Pond) and to help protect vulnerable groundwater resources (where pond design does not consider lining).
- Future pond design must account for changes in the pond's operating conditions throughout its design life. For example in single-level outlet ponds like Linburn, TSV will be reduced by the gradual infilling of the pond if it is adequately performing its sediment removal function. The accumulation of sediment on the bed therefore must be accounted for in flow attenuation calculations.
- Finally, if research suggesting urbanisation can increase peak flows to 5 times greater than the pre-development levels is correct, and retention ponds are subject to inflows from areas that are continually urbanising, then their inflow conditions will not remain stable and, they will have to deal with flows that exceed their design conditions year upon year as development increases. At the very least this should be accounted for at the outset of the development and incorporated into the initial pond designs.

7.2 Design Recommendations

In light of the conclusions drawn from this work, particularly that current design guidance is inadequate for producing ponds that are site-specific and equipped for improving both current and future water quantity and quality, a set of design recommendations have been produced for pond design in the UK. These should be used with care, bearing in mind the success of a SUDS pond is highly dependant upon its site-specific nature. For example, ponds designed for areas in the South of England will have different requirements from those designed for the North-West of Scotland. Furthermore, sites that have a high water table and/or impermeable soils will require different approaches to those that don't.

The design recommendations are illustrated as a flow chart (Figure 7.1). Figure 7.1 describes the design procedure that should be undertaken for stormwater ponds. All of the parameters described in the new procedure could be selected and tested using the pond model presented in this thesis.

It must be emphasised that the site-specific nature of SUDS should be considered and therefore a detailed site investigation is suggested in the recommendations.

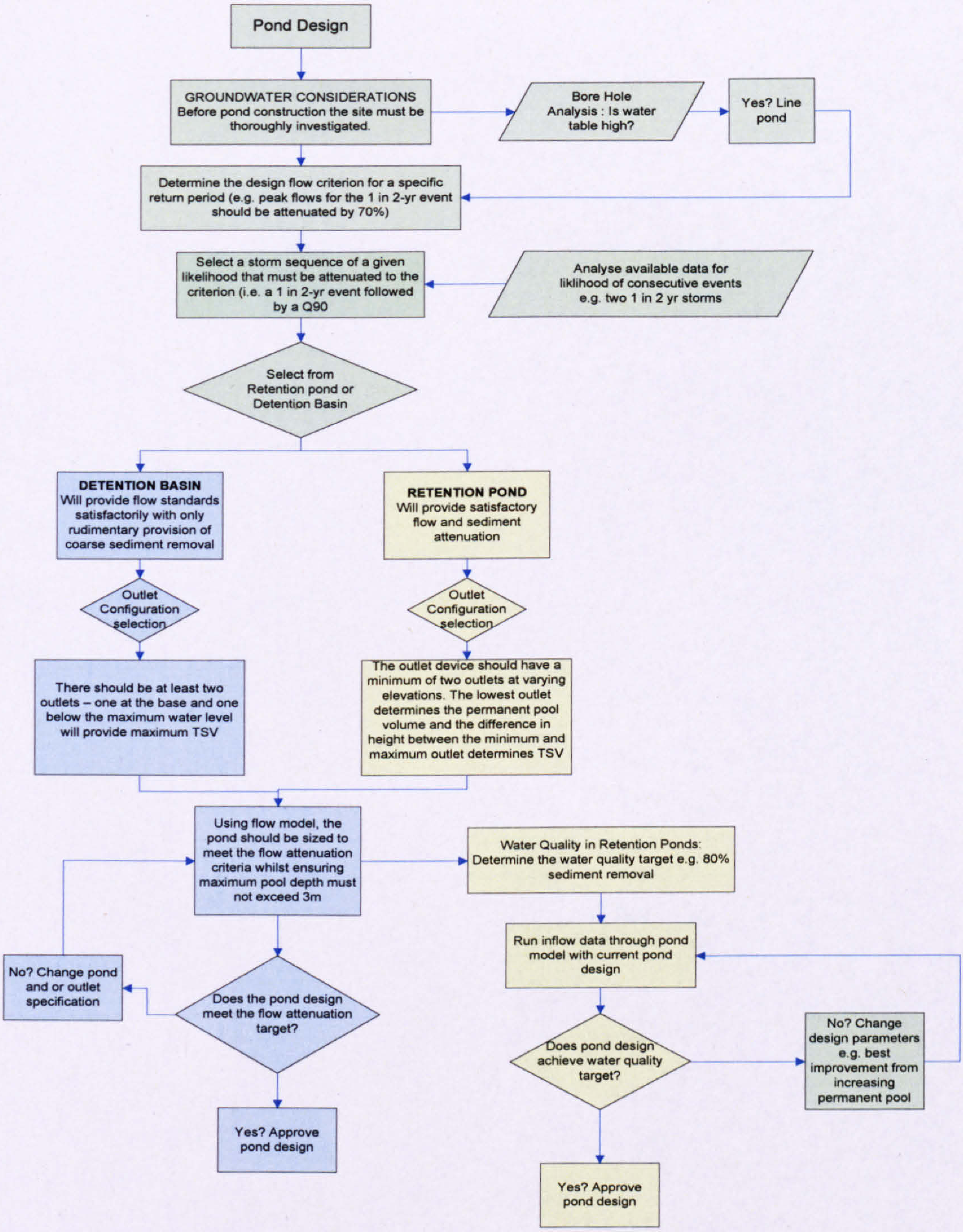


Figure 7.1: Flow chart illustrating proposed design recommendations

Step 1: Before pond construction, a thorough site investigation must be undertaken to ensure that the site is suitable for the pond. In particular, a borehole analysis should be undertaken to determine the proximity of groundwater through out the year. If the water table is high the designer could consider lining the pond. If lining the pond is not feasible then another site should be considered.

Step 2: A design criterion must be determined. In this thesis, a peak flow reduction of 50% was used, whereby peak outflow for the 1 in 25 year storm was reduced to 50% of the peak inflow. Any criterion can be selected, however the design engineer is urged to consider the flow attenuation that will be necessary in the future. Since urban development may continue at the chosen site, the selected criterion should provide adequate flow attenuation over future years as well as at the time of pond design.

Step 3: The thesis has shown that pond design based on a single event storm is inadequate, and therefore designing the pond to attenuate a sequence of storms is required. Analysis of local rainfall records to determine the probability of particular storm sequences (e.g. two 1 in 2 year events, a 1 in 2 year event followed by a Q90 event etc) should indicate which sequences are most likely to pose a risk to flow attenuation performance at the site.

Step 4: The designer should select an appropriate pond type dependant upon user needs. Both detention and retention ponds will offer flow attenuation, and while detention ponds offer some sediment removal, only well-designed retention ponds can offer a high degree of this.

Step 5: The outlet configuration options will vary depending on basin type. A detention basin should have an outlet at an elevation that satisfies the pond depth requirements. However it should also have an emergency overflow pipe at a higher elevation. Findings from this thesis suggest that to minimise the required surface area of a retention pond, it should have a minimum of two functioning outlets at differing elevations. A combination of pipe and weir were used in simulations in this present work; however any outlet type could be used. The lowest outlet elevation will determine the permanent pool depth, while the highest will determine the maximum water level. The height difference between the two outlets will determine the temporary storage volume.

Step 6: With all other pond parameters chosen, the pond must be sized. The site will have a finite size and depending on the use an optimum pond size may already have been recommended by a developer. Using this as a starting point, the pond model can be used to determine if the proposed pond design achieves the flow attenuation target or criterion that has been set. If not, a larger pond radius or different outlet configuration may be required. Note at the design stage consideration should be given to allowing for loss of pond volume due to sediment.

Step 7: When the pond has been sized to achieve the flow attenuation targets, the retention pond can also be designed to achieve particular water quality targets. First, a water quality target must be selected e.g. 80% TSS removal. However, depending on the location of the chosen site, there may be local guidelines for this.

Step 8: Run the inflow data through the water quality module to determine the water quality performance. If water quality performance is poor, parameters such as pipe diameter and permanent pool volume (controlled by the height of the lowest outlet above pond base) can be adjusted, and the model re-run to find the best water quality performance for the pond design. To increase the permanent pool volume, whilst maintaining the flow attenuation performance would require both outlets to be raised simultaneously such that the temporary storage volume remains the same. If sediment removal is still inadequate, the designer may have to go back to step 6 to increase the radius of the pond.

A major flaw in the current guidance is that it uses current data to produce a design storm, with no consideration for possible changes in hydrologic regime. Further, with regard to flow attenuation performance, the current guidelines do not consider multiple storm events. Consequently, two final recommendations are that flow (and/or rainfall) sequences are analysed to determine the likelihood of multiple event scenarios, and that a 20% increase in flow magnitude is assumed for the period 2010-2060. This latter figure may need adjusting as more accurate climate change predictions become available, however, incorporating a 'climate change parameter' into the designs will help to ensure that SUDS designed in the UK will not be obsolete or impracticable in their first few years of operation.

It is recognised that developers and designers may be constrained by factors such as available land and costs. The new set of guidelines provided here, assume that there is available land to increase the size of the pond surface area to meet flow and water quality targets. This is because simulations have shown that sizing the pond to provide enough storage volume is the primary consideration in flow and water quality performance. When design under such conditions is not possible, it is important to remember the other key design parameters that can greatly improve pond performance. To aid design under such difficult conditions a short summary of key design procedures is provided here.

The first step that should be taken is always a thorough investigation of the site – a groundwater borehole analysis should determine whether a liner will have to be considered (this should be carried out at several times in the hydrologic year, particularly winter). If a liner is not required this will reduce the constraints on costs.

If the land available is too small to provide adequate surface area, sufficient TSV may have to be achieved by making the pond deeper. This will be limited by local authority guidelines (CIRIA recommend a maximum depth of 3m for health and safety reasons).

The outlet configuration and the size of the outlet pipe are also essential in ensuring good performance. These must be designed with due consideration for treating a range of storm sizes from the large design storm to the smallest daily storm. This can be achieved by using a multiple outlet device. This thesis considered a dual level device consisting of a low-level pipe and a V-notch weir. However alternatives such as a perforated standing pipe or a hydrobrake may help provide flow control under very difficult design limitations.

Selecting the correct pipe diameter is essential in providing flow control for a range of storms. A pipe diameter that is too small will restrict flows and may lead to the pond filling too quickly resulting in weir flow. A pipe diameter that is too large may allow flows to pass through the pond unrestricted, without any detention at all. The model described in the thesis enables suitable pipe diameters to be selected for a range of storms

Results in this thesis show that a poorly designed pond (i.e. one with inadequate TSVand/or a poor outlet design), such as Linburn pond may provide no flow attenuation benefit at all, and is therefore a waste of resources. If the required TSV still cannot be provided, even under these alternative guidelines, then building a pond at this site may not be appropriate and other flow control and water quality strategies may need to be considered. If TSV cannot be achieved for the design storm, then the addition of other types of SUDS systems (small swales or below ground filter drains etc) around the catchment may need to be considered.

7.3 Future Work

In order to facilitate the better assessment of the performance of SUDS ponds in Scotland, a thorough and well planned monitoring system must be designed. Currently there are no consistent data for specific SUD systems. While the DEX site provides inflow data for some systems, outflow data for others and a selection of snap shot water quality data for the site, there is no consistent long-term approach that will provide the data necessary to make an assessment of the long-term performance of any single system at the site. Furthermore, this ad hoc approach to monitoring prevents predictive modelling based on the kind of site specific data required to make accurate assessments of future performance.

An investigation of the extent to which treatment trains facilitate flow and water quality control in regional SUDS sites should be part of any future work. Quantification of the performance achieved by using multiple systems in series offset against land take and construction costs would provide a useful insight into a more holistic site planning approach.

Despite much of the existing research suggesting a clear relationship between retention time and sediment settling, work conducted here did not identify such a strong correlation. In future modelling studies, a thorough investigation into the mechanisms that influence water quality should be conducted along with an assessment of the optimum method for calculating residence time. This relationship may be dependant upon the assumptions used in determining the predominant type flow in the pond and perhaps different methods of assessing retention ponds will be required for different pond types and shapes. However, this work should facilitate better predictions of sediment capture in retention ponds. Furthermore, a thorough assessment of different methods for sizing ponds for water quality is required after simulations in this current research showed the present method of calculating the treatment volume, Vt to be inadequate.

In all the simulations conducted in the thesis, the outlet device was shown to play an important role in the overall performance of the pond. Although single and multi-level outlets were investigated, due to the availability of data only a very rudimentary investigation of the effect of outlet type (either a pipe or weir) were undertaken in this study. Further work that fully assesses the contributions made by different outlet types such as Hydrobrakes and perforated standing pipes could be undertaken to provide quantification of a range of alternatives for stormwater retention pond design.

Appendix A

Controlled Activities Regulations; proposal made to parliament (Source: Neil McLean, SEPA)

Column 1 = activity	*Column 2 = Rules*
10. Discharge of water run-off from a surface water drainage system to the water environment from construction sites, buildings, roads, yards or any other built developments	(a) the discharge shall not result in pollution of the water environment; (b) the discharge shall not contain any trade effluent or sewage, and shall not result in visible discolouration, iridescence, foaming or growth of sewage fungus in the water environment; (c) the discharge shall not result in the destabilisation of the banks or bed of the receiving surface water; (d) the discharge shall not contain any water run-off from any buildings, roads, yards or other built developments, the construction of which is completed after 1st April 2006, or from construction sites operated after 1st April 2006, unless– (i) those developments or construction sites are drained by a SUD system or equivalent equipped to avoid pollution of the water environment; (ii) the run-off is from a development that is a single dwelling and its curtilage; or (iii) the discharge is to coastal water; (e) the discharge shall not contain any water run-off from– (i) fuel delivery areas and areas where vehicles, plant and equipment are refuelled; (ii) vehicle loading or unloading bays where potentially polluting matter is handled; or (iii) oil and chemical storage, handling and delivery areas; constructed after 1st April 2006; (f) all facilities with which the surface water drainage system is equipped to avoid pollution, including oil interceptors, silt traps and SUD system attenuation,

	settlement and treatment facilities, shall be maintained in a good state of repair; and (g) all reasonable steps shall be t aken to ensure that any matter liable to block, obstruct, or otherwise impair the ability of the surface water drainage system to avoid pollution of the water environment is prevented from entering the drainage system.
11. Discharge into a surface water drainage system.	(a) oil, paint, paint thinners, pesticides, detergents, disinfectants or other pollutants shall not be disposed of into a surface water drainage system or onto any surface that drains into a surface water drainage system; (b) any matter liable to block, obstruct, or otherwise impair the ability of the surface water drainage system to avoid pollution of the water environment shall not be disposed of into a surface water drainage system or onto a surface that drains into a surface water drainage system; and (c) sewage and trade effluent shall not be discharged into any surface water drainage system.

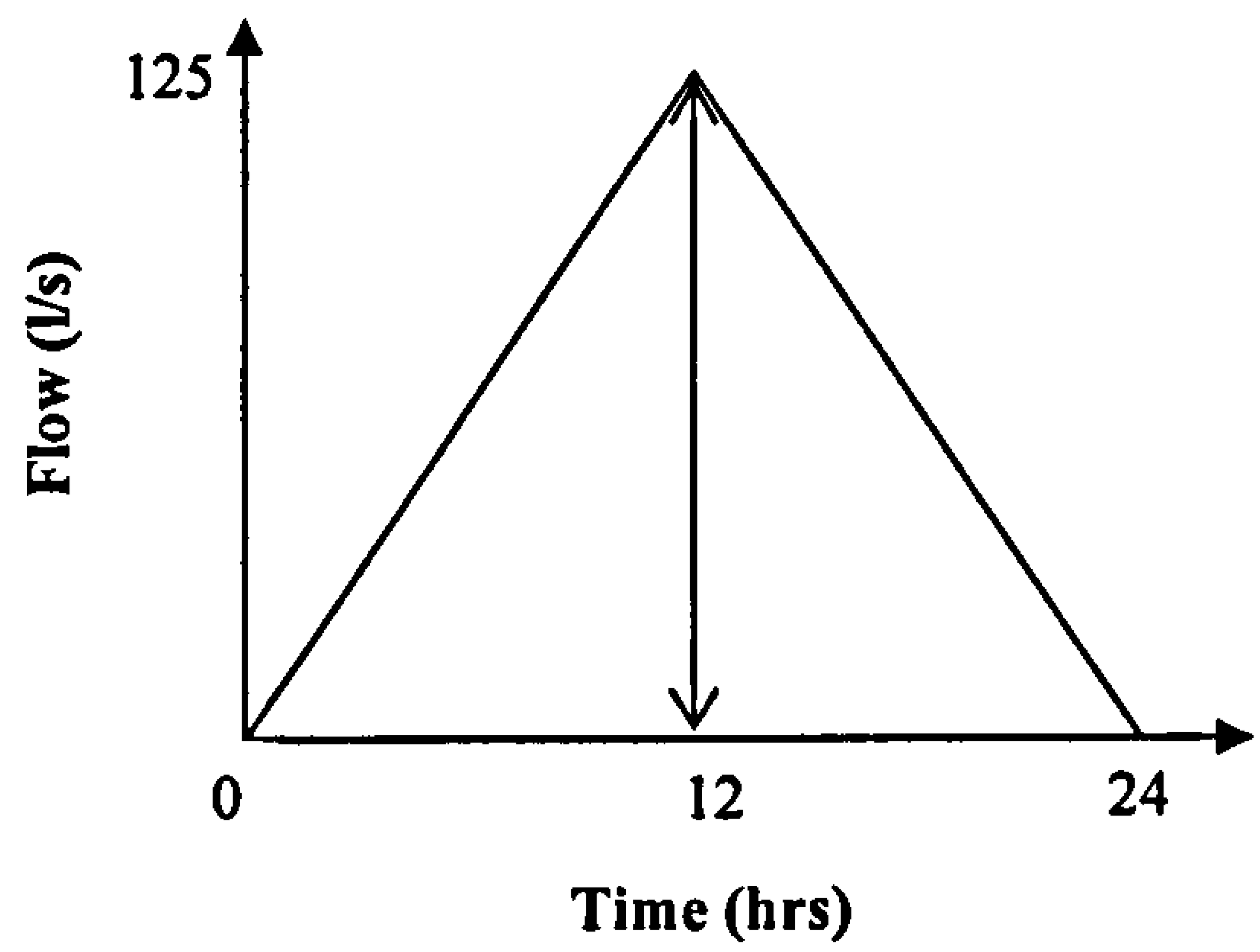
Appendix B

		Hydrology	Hydraulics				Water Quality		Planning	
Program Name	Description (see text for details)	Flows, flood routing	Open channels, waterways	Pipe systems	Culverts, bridges, structures	Storage routing	Pollutant estimation	Water quality controls	BMP evaluation	GIS/ CAD integration
<u>Hydrology only</u>										
RORB	Hydrology	Event				√				
HEC-1/ HMS	Hydrology	Event				√				L
XP-RAFTS	Hydrology	Event				√				L
<u>Hydraulics only</u>										
HEC-RAS	One-dimensional open channel and structure steady-state hydraulics		√		√					√
<u>Hydrology and Hydraulics</u>										
XP-RatHGL	Rational method hydrology, steady-state hydraulics	√		√						√
DRAINS (ILSAX)	Time-area hydrology, storage, pipe or open channel hydraulics	√	√	√		√				L
MIDUSS	Time-area hydrology, storage, pipe or open channel hydraulics	√		√		√				
StormCAD	Time-area hydrology, storage, pipe hydraulics	√		√		√				√
Program Name	Description (see text for details)	Flows, flood routing	Open channels, waterways	Pipe systems	Culverts, bridges, structures	Storage routing	Pollutant estimation	Water quality controls	BMP evaluation	GIS/ CAD integration

Appendix C

The calculation detailed here describes how the size of the catchment is calculated in section 4.2.1 from the peak flow of the storm event.

Based on Figure 4.3, the equivalent 1 in 2 year inflow event has a peak flow of 125l/s at this location. We know from the Tullyallan annual maximum data that the 1 in 2 daily rainfall event is 32mm (BADC) for a 24 hour duration storm.



Total Storm Volume (area of the triangle) $= (24/2)\text{x}60\text{x}60\text{x}125/1000$
$= 5400\ m^3$

If the 1 in 2 daily rainfall is 32mm then using:

$Q_i = Q_d C A$

where, Q_i is the inflow, Q_d is the depth of rain, C is coefficient describing catchment imperviousness (0.1) and A is the area of the catchment, we have:

5400 = 0.032m x 0.1 x Area of Catchment (m^2)

Area of catchment = *1687500m^2 or 1.7 km^2*

Appendix D

Calculating the Treatment Volume for Different Pond Configurations. N.B. The treatment volume should be calculated to reside beneath the lowest outlet in the pond.

The treatment volume is estimated as the runoff generated from the first 15mm of rainfall from the impervious catchment surface (as calculated in appendix C):

Treatment volume = $0.015m \times 0.1 \times 1.7km^2$

$= 2550m^3$

In configuration 1, the detention basin has its weir at 3m, and therefore it is the volume below this elevation that will contain the treatment volume since the other outlet is located at an elevation of 0m (beneath which a treatment volume could not reside). The pond radius required to contain this volume beneath a weir at 3m elevation is:

$$\pi r^2 \times 3 = 2550$$
$$r = \underline{16.45}m$$

In configuration 2, the single-level outlet has a weir at an elevation of 3m. Since it has no other outlet, it is the volume beneath the weir that will contain the treatment volume, and therefore the calculation is as above:

$$\pi r^2 \times 3 = 2550$$
$$r = \underline{16.45}m$$

In Configuration 3, there are two outlets. The weir is set at an elevation of 3m, as in the other two configurations, while the lowest outlet is located at an elevation of 1.5m. Since the treatment volume will reside beneath the lowest outlet, the pond radius required to contain it is:

$$\pi r^2 \times 1.5 = 2550$$
$$r = \underline{23.26}m$$

Appendix E

Published papers

Modelling the flow attenuation performance of retention ponds

S.G. Wallis[1], C.T. Morgan[1], K. Heal[2] and R.J. Lunn[1]

[1]Heriot-Watt University, Edinburgh. [2]University of Edinburgh

INTRODUCTION

Recent initiatives in urban drainage are focused on the prevention of problems as close to source as possible. In the UK the past decade has seen the introduction of Source Control, Best Management Practices and more latterly Sustainable Urban Drainage Systems (SUDS). This trend follows the lead set by developments in the USA, Scandinavia and Australia. The use of SUDS in new urban developments is now required by UK Environmental Regulators, and aims to reduce flood risk, improve water quality in receiving watercourses and provide enhanced amenity value locally. In general terms, SUDS work by storing runoff and simultaneously providing opportunities for the removal of pollutants. Several main types of SUDS can be identified, namely: basins; infiltration devices; swales and porous pavements.

Although the basic principles that determine how SUDS work are known, much less is known about how successful they are in practice, particularly over their entire design life (CIRIA, 2000). Several concerns have been raised, for example: how do systems perform under situations that exceed the design conditions; to what extent is the design performance affected by the build-up of sediment; can pollutants that are attached to settled sediments be released to the overlying water and what is the optimum maintenance strategy required to prevent excessive frequency of failure? To address issues such as these the authors have been studying the performance of SUDS in Scotland using mathematical modelling. The broad aim is to gain an improved understanding of the relationship between design and performance, with an emphasis on identifying the loss of performance that occurs when SUDS operate outwith their design specification. This work complements other study groups in Scotland that are monitoring the actual performance of SUDS, e.g. Jefferies *et al.* (1999).

To date, the authors' work has been targeted at SUDS basins because they have one of the longest histories of use in Scotland. They comprise about 20% of all SUDS constructed in Scotland (McKissock *et al.*, 1999) and serve a variety of urban developments (road systems, housing, retail parks and industrial sites).

In this paper we present some results from the modelling of retention ponds, focusing on flow attenuation issues and the relationship between performance and design, with the aim of quantifying the loss of performance under conditions that exceed the design scenario.

RETENTION PONDS

Operating concepts

There are two basic types of basins used in SUDS engineering, namely retention ponds and detention basins. They both provide both flow attenuation and water quality improvement by the principal action of storing runoff (CIRIA, 2000; Butler and Davis, 2000). By storing water, the passage of storm runoff to a receiving watercourse can be attenuated significantly, i.e. in comparison to the situation where no storage is available, the peak flow magnitude is reduced, the flow hydrograph is distributed over a longer time period and the impact on the watercourse is delayed in time. In addition, the relatively quiescent hydraulic conditions that prevail promote the settling of sediment. As well as reducing the turbidity of the water that leaves the pond, most of the pollutants in the run-off are also removed because they are attached to the sediment particles (Marsalek *et al.*, 1997). In contrast to detention basins, which are designed to be empty at the start of a storm, retention ponds contain a permanent pool of water all year round and store water for longer time periods. Hence, residence times promote increased deposition of sediment. In addition, the ecological habitat that is created provides the conditions for the treatment of pollutants by biochemical processes and by recycling through the biological activity of flora and fauna.

Figure 1 shows the cross-section of an idealised retention pond. It is assumed to be circular in plan, outflow being controlled by a v-notch weir. Let us consider the sequence of events that occurs following the generation of runoff from a single storm event. Clearly, once storm water begins to enter the pond, the water level rises and, until the water level reaches the elevation of the weir crest, there is no outflow and all the water is stored. Once outflow through the weir begins, water continues to be stored above the weir crest elevation until the rate of outflow exceeds the rate of inflow. At this time, the outflow and the water level in the pond have reached their maximum values and the inflow has passed its peak value. We refer to the storage that occurs before outflow begins as *static storage*, and the storage that occurs after outflow begins as *dynamic storage*. As the inflow continues to reduce, the water level falls and the outflow reduces. Some time after the inflow ceases, the water level reaches the weir crest elevation and outflow stops. Assuming there is no further inflow, the water level falls very slowly over the following days, principally through evaporation and uptake by in-pond flora. If the pond does not have an impermeable liner, water can also be exchanged with the ground. Hence, the final water level in the pond may be controlled by the local water table.

It is important to realise that the length of time between storms, combined with the time taken for the water level to fall, has an important bearing on the ability of the pond to store the inflow (Guo, 2002). Clearly, if the water level at the beginning of a storm is close to the weir crest elevation, much poorer attenuation will occur than if the initial water level is close to the design elevation for the water surface of the permanent pool. Thus the static storage is an important parameter. The role of the dynamic storage is, perhaps, less obvious. This is controlled by the angle of the v-notch weir. If this is small, outflow increases slowly and water level increases quickly compared to the case when the weir angle is large. As a result, small weir angles promote good flow attenuation because a large inflow volume can be retained and it can be retained for a longer period of time. However, there is an increased risk of the pond becoming full and overtopping.

It is clear, as a general principle, that increasing either or both of the static and dynamic storage will increase flow attenuation. What is far from clear, however, is by how much flow attenuation will be improved by a given increase in either type of storage. Perhaps of more importance, however, is by how much will attenuation be reduced if (a) the expected static storage is not available at the start of a storm, and (b) how strong an influence does the choice of outflow weir angle have on the potential flow attenuation provided by a pond.

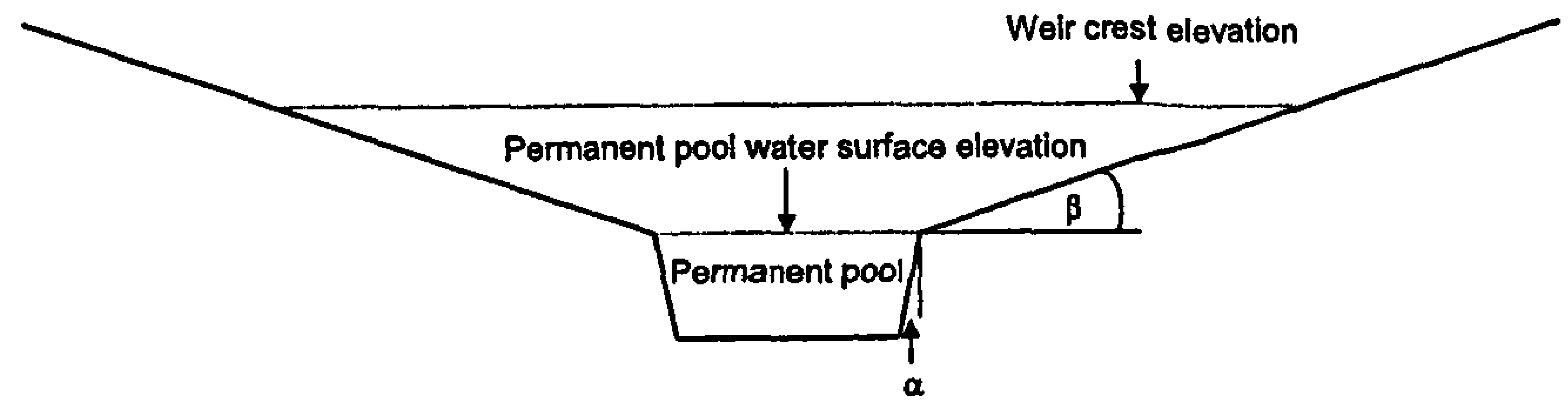


Fig. 1 Cross-section of circular retention pond

THEORETICAL CONSIDERATIONS

The flow dynamics of SUDS basins can be described by the following lumped mass balance equation:

$$\frac{dV}{dt} = Q_i - Q_o \tag{1}$$

where V is the storage (or volume of water) in the basin, Q_i is the inflow to the basin and Q_o is the outflow from the basin. This is the familiar storage routing equation that is used frequently in hydrological studies, and which states that the rate of change of storage is equal to the difference between inflow and outflow. Figure 2 shows a typical response predicted by Equation 1, in which for reasons of simplicity the inflow hydrograph is assumed to be an isosceles triangle. The key feature illustrated by the figure, however, is that (irrespective of the shape of the inflow hydrograph) the peak outflow occurs when the outflow equals the inflow. This implies that all solutions for the peak outflow lie on the recession limb of the inflow hydrograph and that, therefore, there is a simple relationship between the peak outflow and its timing. What is not clear, however, is how the static and dynamic storages determine any particular solution for the magnitude and timing of the peak outflow, although it is evident that increasing the storage results in a reduced peak outflow magnitude occurring at a later time. It can also be noticed that the time delay between the starts of the outflow and inflow hydrographs, respectively, is directly related to the static storage because outflow does not start until the inflow has occupied the static storage.

Below we discuss the results of numerical simulations of Equation 1 that were designed to further elucidate the attenuation performance of retention ponds. In particular, the

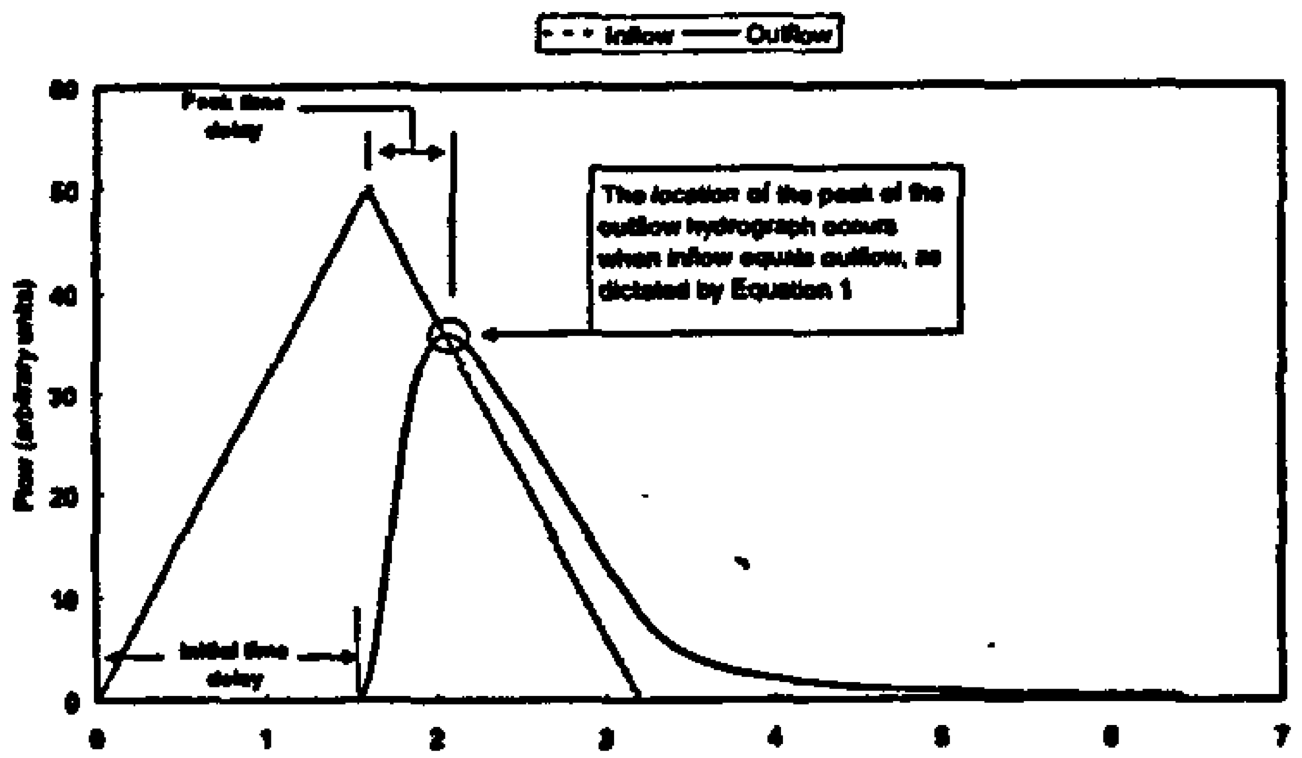


Fig. 2 Response of a retention pond subject to a triangular inflow hydrograph

results enable the following sort of questions to be addressed:

1. How is the peak outflow and its timing related to static storage and outflow weir angle?
2. By how much does the attenuation performance of a retention pond decrease as deposited sediment accumulates?
3. By how much does the attenuation performance of a retention pond decrease if, at the start of a storm, the water level is above the design elevation for the water surface of the permanent pool?
4. By how much does the attenuation performance of a retention pond decrease as the inflow volume increases?

The last three questions are of particular significance to designers of retention ponds, since they throw light on the loss of performance that can be expected under conditions that exceed the design scenario.

NUMERICAL SIMULATIONS

The numerical solutions were based on level pool routing (Maidment, 1993) and used the simplest numerical technique, namely the explicit Euler method. Hence Equation 1 was approximated by:

$$\frac{V^{n+1} - V^n}{\Delta t} = Q_i^n - Q_o^n \qquad (2)$$

where superscript n refers to known values of variables at time n and superscript n+1 refers to corresponding unknown values at a time interval, Δt, later. At each time step in the simulation: Equation 2 was solved for V^{n+1}, the corresponding water level was evaluated from the geometry of the pond and the corresponding outflow was evaluated from the stage-discharge equation for a v-notch weir.

Various simulations were carried out for the basic configuration illustrated in Figure 1, assuming no exchange with groundwater. In all cases $\alpha = 0°$ so that the permanent pool was cylindrical, and bδ took values between 90° and 4°. Thus a range of geometries was covered that extended between a purely cylindrical basin (typical of what might be constructed underground) and a basin having a gently sloping upper portion (more typical of the majority of retention ponds in use). Inflows consisted of isosceles triangular hydrographs having a range of peak flows and durations (and hence total inflow volumes). Simulations were carried out for six weir angles in the range 45° – 160° and for a range of static storages, determined by various combinations of permanent pool radius, weir crest elevation and initial water level. Following an analysis of the convergence of the simulations as a function of time step size, a time step of 1.5 minutes was used in all the simulations.

INTERPRETATION OF RESULTS

Initially, the attenuation performance of the ponds was explored by examining the following performance indicators: total outflow volume, peak outflow, peak time delay and initial time delay (see Figure 2 for definitions). Qualitatively, the results can be summarised as follows:

- As expected, the total outflow volume was simply the difference between the total inflow volume and the static storage available at the start of the inflow hydrograph.
- For a fixed inflow hydrograph it was found that:
  – the peak outflow reduced with increasing static storage and with decreasing weir angle (as argued earlier)
  – both time delays increased with increasing static storage, which reflects the increasing time required for the inflow to occupy the storage available
  – decreasing the weir angle increased the peak time delay but had no effect on the initial time delay, which reflects the control exercised by the weir only once outflow begins
- For a fixed pond, increasing the inflow volume led to larger total outflow volumes and peak outflows and smaller time delays, which reflects a reduction in the influence of the static storage under high inflow volume conditions.

To study the relationship between performance and design alluded to earlier, a quantitative analysis of the simulations was undertaken for ponds having a fixed permanent pool radius. The study focused on the peak outflow and its timing. In this, it was useful to first normalise the inflow and outflow hydrographs by considering the following non-dimensional variables: peak flow/peak inflow, and time of peak flow/time of peak inflow. Using these non-dimensional co-ordinates, all inflow hydrographs are the same and all peak outflows lie on the recession limb of the non-dimensional inflow hydrograph.

Simulation results from a cylindrical pond of radius 5 m are shown in Figure 3. These illustrate how static storage and weir angle combine to dictate the particular attenuation provided by a specific pond. The continuous line on the figure is the recession limb of the non-dimensional inflow hydrograph, which can be interpreted as the domain of all possible solutions for the peak outflow. Hence, as explained earlier, all the simulations fall on this line. Indeed, the simulations appear in four groups, each of which represents a different value of static storage. On the figure the static storage is represented as a percentage of the total inflow volume and values are indicated by the legend: for example, a value of 63% corresponds to a case where 63% of the total inflow volume is stored as static storage. Note that the location of these groups within the domain of all possible solutions is directly related to the magnitude of the static storage. Within each group results are shown for six values of weir angle. In all the groups the weir angles appear in the same order, with the smallest at the bottom-right and the largest at the top-left of each group. The smallest and largest values are shown on one of the groups.

The results in Figure 3 illustrate the theoretical trend introduced earlier, namely that a large static storage and a small weir angle promote better attenuation performance. In general, attenuation performance is more sensitive to the static storage than it is to the weir angle, although the weir angle has more influence as static storage increases. It is also evident that at least 50% of the inflow needs to be stored in a cylindrical pond to achieve even a modest, say 20%, decrease in peak outflow.

Focusing now on the peak outflow and its relationship to the volume of the inflow stored, the same data are re-plotted in Figure 4. This shows the non-linear nature of the relationship between the non-dimensional peak outflow and the static storage, expressed as a percentage of the total inflow volume, as before. The figure illustrates again the relatively weak role played by the weir angle and, importantly, enables the loss of performance due to reduced availability of static storage to be estimated. For example, a reduction in the percentage of

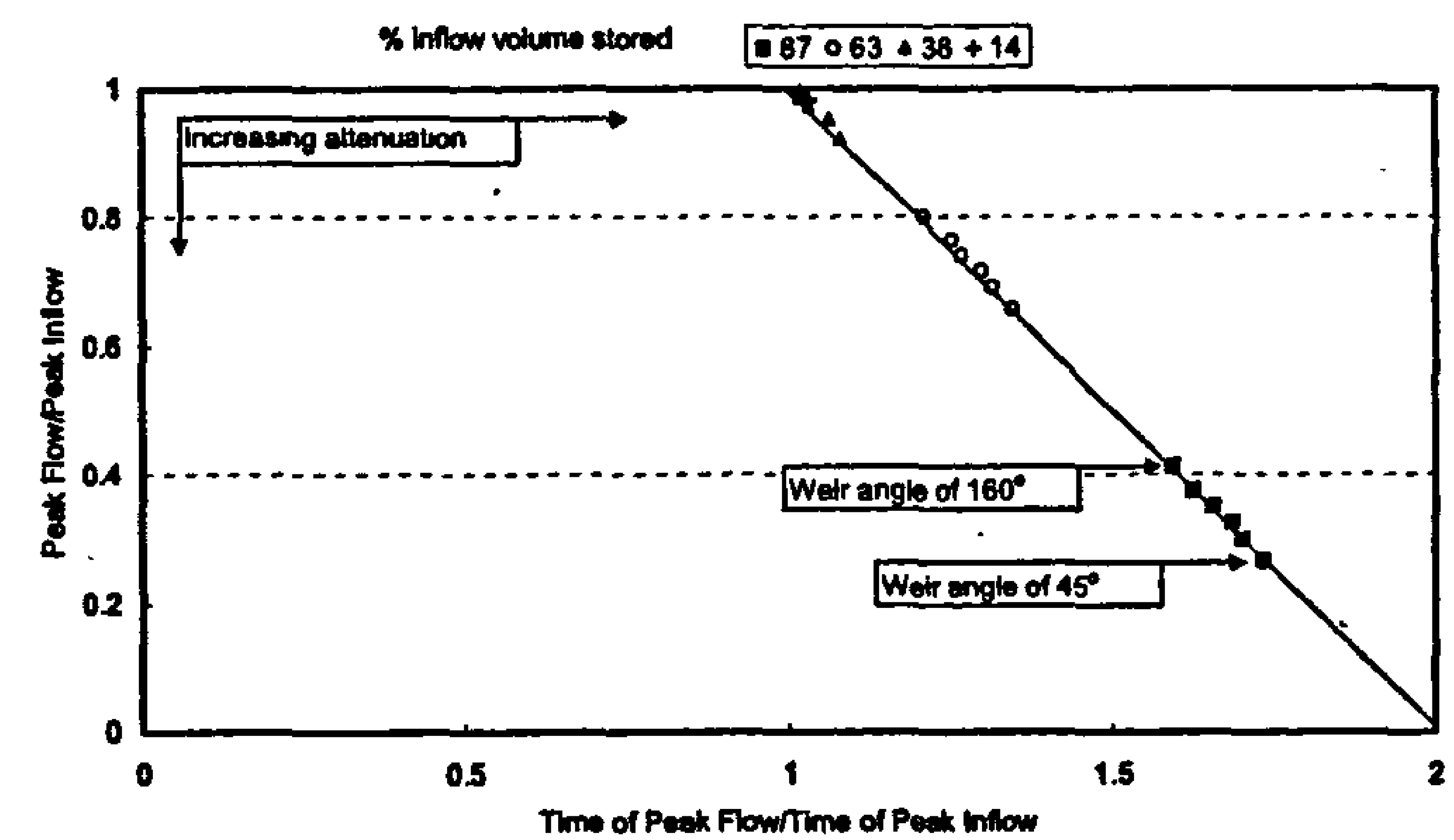


Fig. 3 Non-dimensional flow attenuation of cylindrical ponds as function of static storage and weir angle

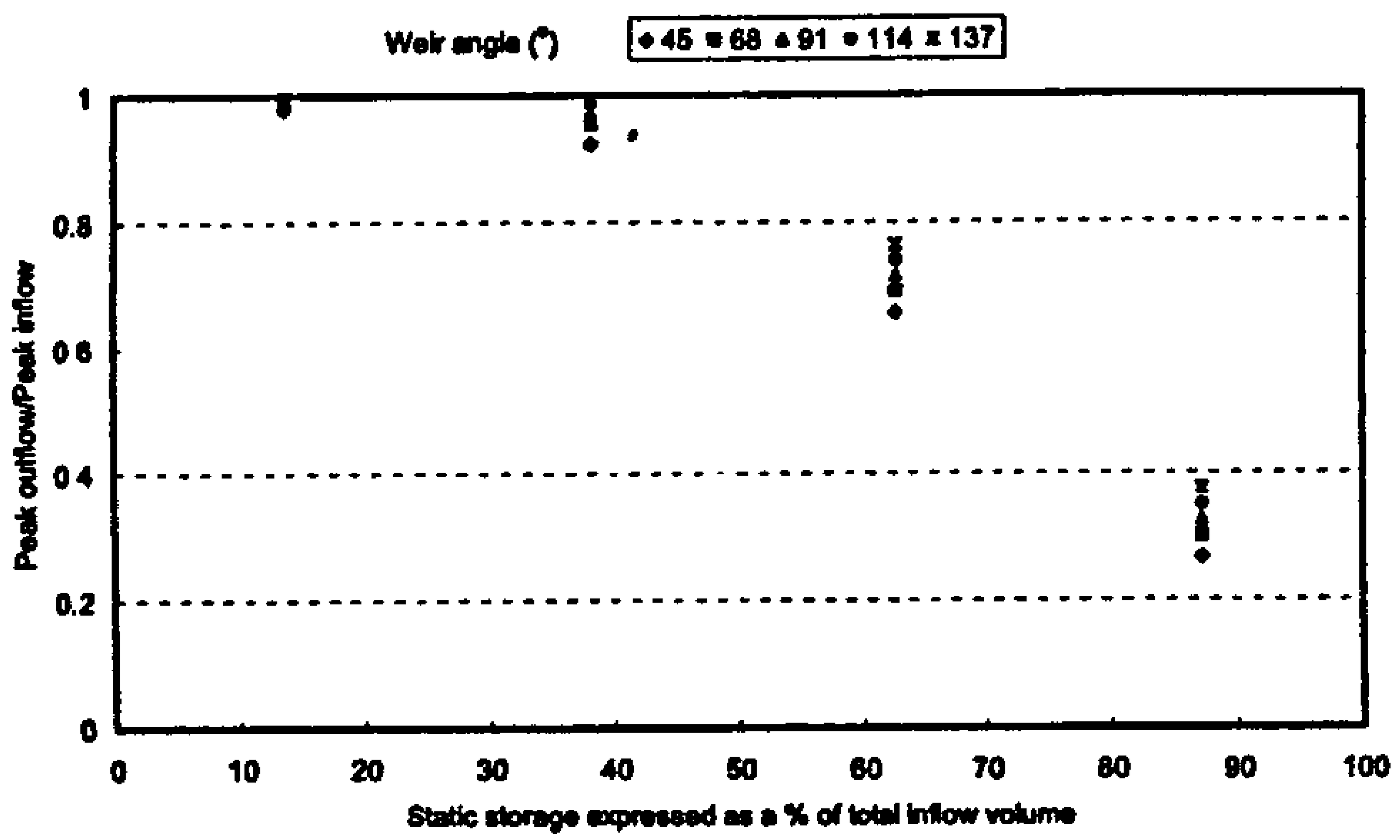


Fig. 4 Non-dimensional peak outflow for a cylindrical pond as a function of static storage and weir angle

total inflow volume stored from 90% to 70% causes a doubling of the peak outflow. In contrast, any reduction in static storage below 50% has relatively little impact — the performance being poor throughout this range, of course.

Figure 5 illustrates the impact of total inflow volume on the attenuation performance of the same cylindrical pond considered in Figure 4, but just for one weir angle. Simulations were carried out for several total inflow volumes. To illustrate the change in attenuation performance in a design context, it was assumed that this pond had been designed to provide a peak outflow of 72% of the peak inflow, which required that 63% of the total inflow volume was stored as static storage. The data labels on the figure show the ratio of total inflow volume to the design total inflow volume for all the simulations. Thus, the data label of 1.0 indicates the design condition for which the peak flow ratio is 0.72 and, for

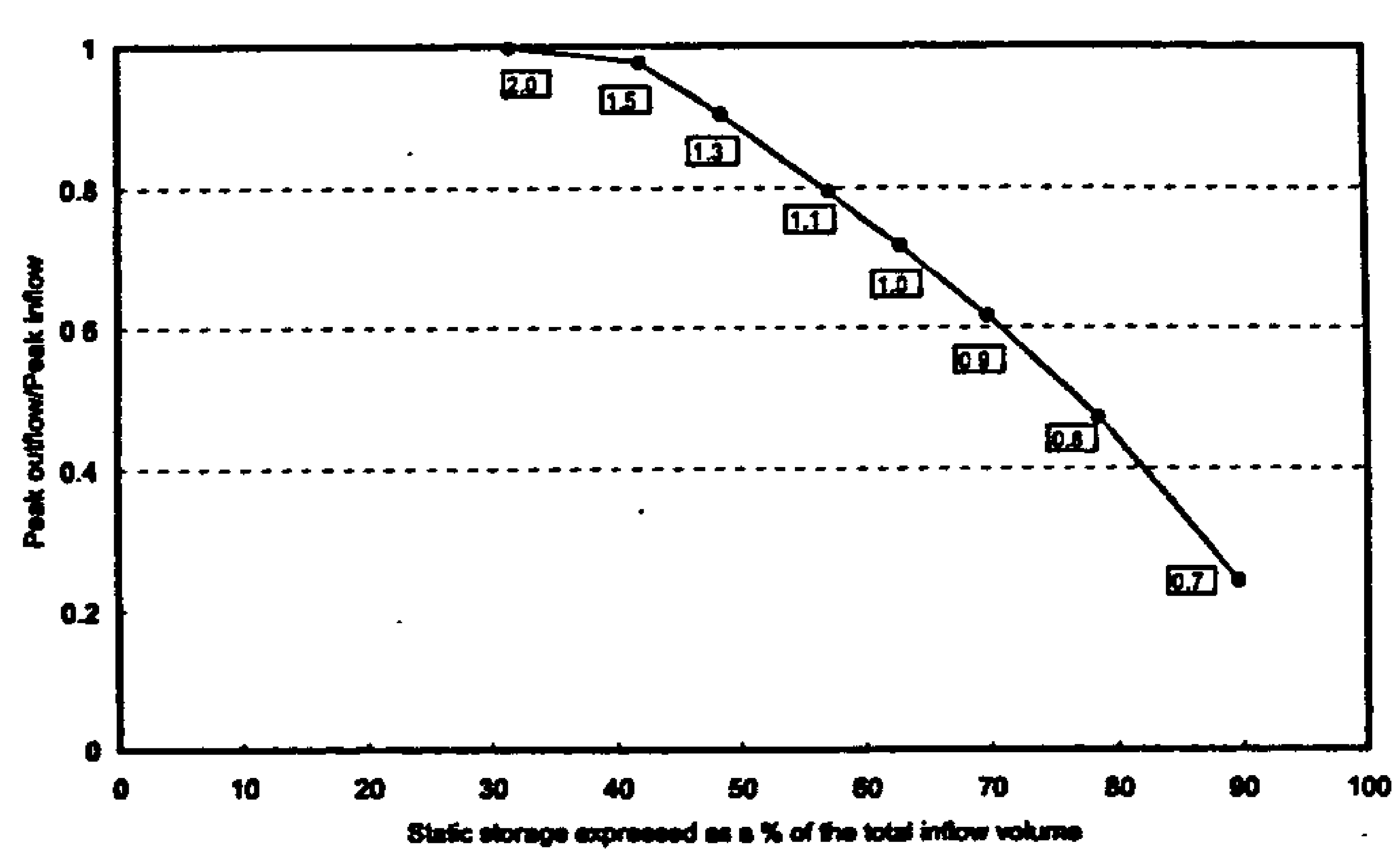


Fig. 5 Influence of inflow volume on attenuation performance for a cylindrical pond. the data labels indicate different inflow volumes (see text)

example, the data label of 1.3 indicates the performance that the same pond would provide if the inflow volume were 30% larger than the design inflow (the peak flow ratio is now 0.90, indicating a 25% increase in peak outflow (100 × (0.90 – 0.72)/0.72). Notice that a doubling of the inflow volume yields a peak flow ratio of 0.997 indicating that the pond would provide no attenuation. Similarly, the data label of 0.7 indicates by how much better the pond performs when the inflow volume is 70% of the design inflow volume.

Figure 6 illustrates the impact of total inflow volume on the attenuation performance of three conical ponds. The ponds that were simulated had the same (cylindrical) permanent pool as the pond used above, but they had different upper side slopes defined by three values of β (15°, 8°, 4°), see Figure 1. For each value of β, simulations were carried out for the same range of total inflow volumes and the results are presented in the same way as in Figure 5. The two data labels indicate the maximum and minimum ratios of inflow volume to design inflow volume that were used. The purely cylindrical case (β = 90°) is also shown in Figure 6 for reference purposes. These results show three important things. Firstly, attenuation performance improves as β decreases. This is shown by the movement of the trend line towards the bottom left-hand corner of the figure as β increases. Secondly, and more specifically, for the same inflow volume a significant reduction in peak outflow is achieved by a modest increase in static storage as β increases. For example, the circled symbols correspond to the same total inflow volume for each pond: these show the ratio of peak outflow to peak inflow reducing from 60% to 49% to 27%, corresponding to static storages of 65%, 68% and 72%, respectively, as β decreases. This reflects the well-known idea that increasing the surface area of a pond is the most effective way of improving its performance, i.e. it is not only the magnitude of the static storage that is important, but the way in which it is distributed plays a significant role. Thirdly, ponds with shallower side slopes are capable of providing more significant attenuation over a much larger range of inflow volumes than ponds with steeper side slopes.

APPLICATION OF THE RETENTION POND MODEL FOR PREDICTION OF LONG-TERM SUDS PERFORMANCE IN SCOTLAND

As discussed in the introduction, the central aim of this research project is to develop a model to assess the performance of retention ponds in Scotland over their entire design life, during which there are likely to be occasions when conditions lie outwith the design scenario. For example, if deposited sediment is not removed on a regular basis, then the available static storage will inevitably reduce over time. Similarly, run-off patterns are likely to alter either due to climate change or to further urbanisation. This could cause increasing inflow volumes over time and, if the annual distribution of events changes, then in addition retention ponds may not be fully drained at the start of storm events, effectively

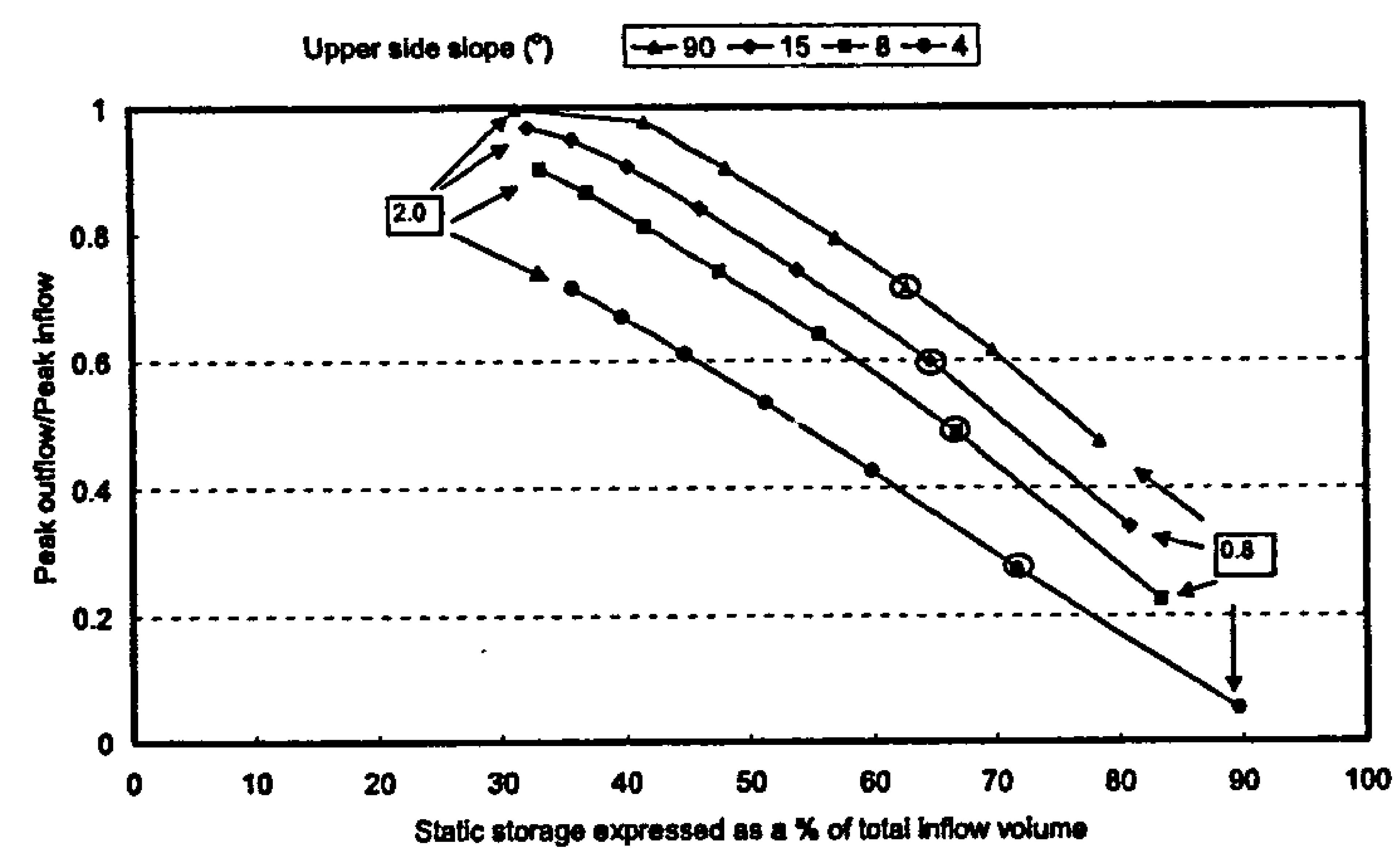


Fig. 6 Influence of pond shape and inflow volume on attenuation performance. the data labels indicate different inflow volumes (see text)

reducing the static storage further. Figures 4,5 & 6 provide the sort of information that could be used to accomplish this. At the same time, the results presented here serve to verify the model (for example, all the peak outflows from all the simulations lie on the continuous line in Figure 3, as they ought to). Furthermore the model has been validated using inflow and outflow records from existing ponds (not presented here).

Research is now in progress to develop a scenario-based modelling approach that can be used to investigate long-term Scottish pond performance. These scenarios will investigate the relationship between climate, surrounding land use, groundwater, and pond performance in terms of both flow attenuation and outflow water quality. For example, work is currently focused on an initial scenario of long-term prediction of outflow water quality. Here, daily rainfall records from the early 1900s to the present day are being analysed to determine the length distribution of antecedent periods and the severity of subsequent rainfall events. The pond model will then be applied, using a variety of assumptions for the surrounding drainage basin, to investigate the significance of the length of the antecedent period and the subsequent rainfall event for outflow water quality. It is anticipated that a long antecedent period will allow an accumulation of sediment in the drainage area that can then be mobilised by a high intensity rainfall event. Since the design life of a pond may be 30 to 40 years, any evidence of climate change in the rainfall records will also be investigated.

CONCLUSIONS

The results presented here demonstrate some general principles relating to the flow attenuation performance of circular retention ponds that have outflows controlled by v-notch weirs. The static storage available at the start of a storm, and the way it is distributed, are the most significant factors that affect performance. As well as agreeing with established design principles, these results also quantify the change in performance that occurs under different static storages and inflow volumes. Hence the results indicate how the performance of existing ponds will deteriorate if the design static storage is not available due, for example, to the accumulation of sediment over time or if the time interval between storms is insufficient for the water level to fall to the design elevation of the permanent pool. Similarly, with a view to climate change issues, the results indicate how existing ponds might under-perform if runoff volumes increase due to changing rainfall patterns. The results also show that the role of the v-notch weir is not particularly significant.

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Using Pollutant Transport Modelling to Improve the Design of Stormwater Detention Ponds

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ABSTRACT

This paper concerns the mathematical modelling of stormwater detention ponds, with an emphasis on flow attenuation and the dilution of soluble pollutants. A model was developed that treats detention ponds as deterministic, lumped hydrological systems and in which governing mass conservation equations are solved using standard numerical methods. The model output consists of an outflow hydrograph and an outflow pollutograph, given the corresponding inflow conditions, the pond geometry and the pond outlet device configuration. Eighteen simulations of solute transport were undertaken in order to identify some of the major influences on solute concentration in the outflows from detention ponds. The simulations covered a range of inflow scenarios, outlet configurations and initial conditions. An analysis of the flow and solute concentration results suggests that good attenuation of the peak outflow occurs when the pond is relatively empty at the start of an inflow event, whilst good dilution of soluble pollutants in the inflow occurs when the pond is relatively full. It is difficult to simultaneously satisfy both these requirements. Consequently, for example, "first flush" pollution events are not diluted particularly well if the pond is empty at the start of the event. Further work is required to understand how pond designs might be improved to ameliorate such conflicts, and this will need to consider the role of inter-event periods.

1. INTRODUCTION

Stormwater detention ponds are an established part of the urban drainage engineer's portfolio of measures for ameliorating the environmental impact of surface water runoff in urban areas (CIRIA, 2000). In simple terms, the storage provided by a pond attenuates the flow, reducing the peak outflow and delaying the movement of the water further down the catchment. In addition, the quality of the water that is released to a watercourse via some outlet device continues to improve the longer the stormwater is detained, because suspended material has a greater opportunity to be removed by settling under gravity. Thus not only is the appearance of the released water enhanced through a reduction in turbidity, but since many of the major pollutants carried by surface runoff are attached to sediment particles, the potential polluting effect of the water (Ellis & Hvitved-Jacobsen, 1996) is also reduced.

However, the settling mechanism does not remove dissolved pollutants from the stormwater. Indeed, the only physical mechanism that is available for reducing the concentration of solutes is the dilution available from the mixing of the inflow to a pond with the water that is already stored there. Clearly, in some cases chemical and biological processes may be taking place, however, the timescales required for these to reduce pollutant concentrations significantly are generally longer then the physical detention times. An obvious exception to this, of course, is where retention ponds are used, for which (by design) the stormwater is stored for a much longer period of time (CIRIA, 2000).

In this paper the results of some numerical simulations of solute transport through a detention pond are described. The simulations were designed to identify some of the major influences on solute concentration in the outflows from detention ponds. The work is part of a broader modelling study of the flow attenuation, solute transport and sediment transport characteristics of detention ponds and retention ponds.

2. MODEL DEVELOPMENT

The mathematical model consists of two components: a flow model and a solute transport model. The model was originally developed to cater for a generic detention pond or retention pond geometry that consists of two truncated conical elements. In this paper only detention ponds are considered and a simplified geometry consisting of one truncated conical element is used. Figure 1 illustrates the generic detention pond under consideration. In both the flow and the solute transport models the pond is modelled as a deterministic, lumped system. The models are described below.

2.1 Flow Model

Flow through the pond is modelled using a standard storage routing method (Mays, 2001; Shaw, 1994) that is based on the following equation that describes the conservation of mass of water:

$$\frac{dV}{dt} = Q_i - Q_o \quad (1)$$

where V is the volume of water in the pond (often termed storage), Q_i is the volumetric flow rate of water entering the pond (inflow) and Q_o is the volumetric flow rate of water leaving the pond (outflow). Equation (1) is solved to give the temporal variation (hydrograph) of outflow, assuming that the inflow hydrograph is known. The outflow passes through one or more outlet devices that are described by hydraulic head-discharge equations. Thus the outflows are actually calculated from predicted water levels in the pond. Two types of outlet devices are considered here: a submerged horizontal pipe located at the base of the pond (modelled as a submerged orifice) and a v-notch weir located at a specified elevation above the base of the pond. The hydraulic equations for these two devices are (Chadwick & Morfett, 1998):

(a) submerged pipe

$$Q_{sp} = aC_D\sqrt{2gH} = K_{sp}H^{0.5} \quad (2)$$

where Q_{sp} is the flow through the pipe, a is the cross-sectional area of the pipe, C_D is the coefficient of discharge, g is the acceleration due to gravity, H is the head and K_{sp} is a constant;

(b) v-notch weir

$$Q_{vnw} = \frac{8}{15}C_D\sqrt{2g}\tan(\gamma/2)H^{2.5} = K_{vnw}H^{2.5} \quad (3)$$

where Q_{vnw} is the flow through the weir, γ is the weir angle, K_{vnw} is a constant and the other symbols are as previously defined.

The values of the coefficients of discharge for these two devices are not necessarily the same, although in the simulations described later, the same value was used for both. Similarly, whereas the head for the submerged pipe is simply equal to the water level in the pond, the head for the v-notch weir is the difference between the water level and the weir crest elevation, y_{vnw}. Clearly, outflow through the pipe occurs whenever there is water in the pond, but outflow through the weir only occurs when the water level in the pond is above the weir crest. The total outflow, Q_o, is simply the sum of the pipe and weir outflows.

Modelling the pipe outlet using equation (2) implies certain assumptions that may not be true at all times in practice. For example, the equation assumes that the outlet behaves as a small orifice that is controlled solely by inlet conditions (Urbonas & Stahre, 1993; Mays, 2001). If the head is low or if the pipe becomes surcharged, equation (2) would be invalid. If the pipe were long, then there would be additional friction losses to be accounted for. Nevertheless, equation (2) has the attraction of simplicity.

Equation (1) can be solved in several ways. Since inflow hydrographs can rarely be represented by a simple function, a numerical approach is usually adopted. Traditionally, hydrologists have employed the storage-indication method (Mays, 2001; Shaw, 1994) although there are many standard numerical algorithms for ordinary differential equations that could be used instead (Chapra & Canale, 1998; Mason & Stocks, 1987; Quinney, 1987; Griffiths & Smith, 1991). The latter route is followed here mainly because these methods can also be used for the solute transport model.

Expressing equation (1) in a standard time-weighted finite difference form gives:

$$\frac{V^{n+1} - V^n}{\Delta t} = \theta(Q_i - Q_o)^{n+1} + (1-\theta)(Q_i - Q_o)^n \quad (4)$$

where superscripts n+1 and n refer to values evaluated at two times separated by the time step, Δt, and θ is a time weighting parameter ($0 \le \theta \le 1$). It can be assumed that any term evaluated at time n is known, so that only terms evaluated at time n+1 need to be calculated. The left-hand side of equation (4) represents the change in volume that occurs during the time step. This can be expressed in terms of the surface area of the pond, A, and the water level, y, as follows:

$$V^{n+1} - V^n = \tilde{A}\left(y^{n+1} - y^n\right) \quad (5)$$

where the ~ overbar indicates an average value during the time step. Combining equations (4) and (5) gives:

$$\frac{\tilde{A}\left(y^{n+1} - y^n\right)}{\Delta t} = \theta(Q_i - Q_o)^{n+1} + (1-\theta)(Q_i - Q_o)^n \quad (6)$$

which can be re-arranged to give:

$$y^{n+1} = y^n + \frac{\Delta t}{\tilde{A}}\left[\theta(Q_i - Q_o)^{n+1} + (1-\theta)(Q_i - Q_o)^n\right] \quad (7)$$

or:

$$y^{n+1} = B - \frac{\Delta t\theta}{\tilde{A}}Q_o^{n+1} \quad (8)$$

where:

$$B = y^n + \frac{\Delta t(1-\theta)}{\tilde{A}}(Q_i - Q_o)^n + \frac{\Delta t\theta}{\tilde{A}}Q_i^{n+1} \quad (9)$$

All the terms on the right-hand side of equation (9) are known, either because they are all evaluated at time n or, in the case of the third term, because all values of the inflow are known. Replacing the unknown outflow in equation (8) by the sum of equations (2) & (3) gives:

$$y^{n+1} = B - \frac{\Delta t\theta}{\tilde{A}}\left(K_{sp}\left(y^{n+1}\right)^{0.5} + K_{vnw}\left(y^{n+1} - y_{vnw}\right)^{2.5}\right) \quad (10)$$

Generally, equation (10) is non-linear in y^{n+1} and needs to be solved via iteration. When $\theta = 0$, however, a direct or explicit solution is possible. This corresponds to the Euler algorithm (Chapra & Canale, 1998; Mason & Stocks, 1987; Quinney, 1987; Griffiths & Smith, 1991). It is well known that the simplicity of this algorithm needs to be balanced against its relatively poor accuracy and potential for instability. Taking ($0 < \theta \le 1$), however, though incurring extra computational cost, yields a more robust solution. Following standard practice, $\theta = 0.5$ (corresponding to the trapezium method (Quinney, 1987; Griffiths & Smith, 1991) or the Crank-Nicolson method (Griffiths & Smith, 1991)) was adopted. This gives a theoretically more accurate solution than other values of θ, and the algorithm has good stability characteristics. Again, following standard practice, Newton-Raphson iteration (Chapra & Canale, 1998; Griffiths & Smith, 1991) was used to cater for the non-linear nature of equation (10).

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Finally, the surface area of the pond was expressed in terms of the water level using (see Figure 1):

$$A = \pi\left(\frac{D}{2} + \frac{y}{\tan\alpha}\right)^2 \tag{11}$$

where D is the diameter of the pond at its base and α is the side slope of the pond. To simplify the detail of the Newton-Raphson iteration, the surface area used in equations (9) & (10) was the area corresponding to y^n. To maintain the highest accuracy, this surface area could, instead, have been evaluated as the average of the areas corresponding to y^n and y^{n+1}, however, such a refinement was not considered to be necessary because it was anticipated that relatively small time steps would be used in the simulations.

2.2 Solute Transport Model

The solute transport model is based on a similar equation to the flow model, but in this case the equation describes the conservation of mass of solute:

$$\frac{d(VC)}{dt} = Q_i C_i - Q_o C_o \tag{12}$$

where C is the concentration of the solute in the pond, C_i is the solute concentration in the inflow and C_o is the solute concentration in the outflow. Equation (12) is solved to give the temporal variation of C_o, assuming that volumes and flows are known from the flow model and that the solute concentration in the inflow is known. Using a standard time-weighted finite difference form, as before, gives:

$$\frac{(VC)^{n+1} - (VC)^n}{\Delta t} = \theta(Q_i C_i - Q_o C_o)^{n+1} + (1-\theta)(Q_i C_i - Q_o C_o)^n \tag{13}$$

Assuming that the pond is well mixed, so that solute that enters the pond is instantaneously and uniformly mixed throughout the water in the pond, then C_o can be replaced by C. Thus the pond is assumed to behave as a Continuously Stirred Tank Reactor (Chapra, 1997; Mihelcic, 1999). After some re-arrangement, equation (13) becomes:

$$C^{n+1}(V + \theta\Delta t Q_o)^{n+1} = C^n(V - (1-\theta)\Delta t Q_o)^n + \theta\Delta t(Q_i C_i)^{n+1} + (1-\theta)\Delta t(Q_i C_i)^n \tag{14}$$

Hence, C^{n+1} is found easily since all the other terms are known and there are no non-linearities. As before taking $\theta = 0.5$, enhances the accuracy of the calculation.

2.3 Simulations

The simulations of solute transport through detention ponds were designed to identify some of the major influences on solute concentration in the outflows from detention ponds. The geometry of the pond, the location and characteristics of the v-notch weir, the location of the submerged pipe and the inflow hydrograph remained fixed whilst the pipe diameter and the initial water level in the pond were varied. Also, different temporal distributions of solute (pollutographs) in the inflow were considered.

Mainly for reasons of simplicity, the pond was assumed to be cylindrical (ie $\alpha = 90°$, see Figure 1), with a radius of 10m. The crest of the v-notch weir was located 2m above the base of the pond, the weir angle was specified as 90° and the coefficient of discharge was specified as 0.6 (Chadwick & Morfett, 1998). The pipe was located at the base of the pond, its coefficient of discharge was also specified as 0.6 (Chadwick & Morfett, 1998) and three diameters were considered: 0.2m, 0.1m and 0m, the latter corresponding to there

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being no pipe at all. The inflow hydrograph was an isosceles triangle in shape of duration 3.2 hours and peak flow of 175l/s. Thus the total inflow volume was $1008m^3$, compared to maximum available storage (below the weir crest) of $628m^3$. Two initial water level conditions were considered: empty and full (ie at the weir crest elevation), corresponding to long and short times between successive storm events.

Three pollutographs were specified, as shown in Figure 2. In Case (a) the solute concentration was directly correlated with the magnitude of the inflow; in Case (b) there was a short pulse of solute during the rising limb of the inflow hydrograph; in Case (c) the same short pulse of solute occurred during the falling limb of the inflow hydrograph. Case (a) corresponds to a continuous release of a pollutant during a storm event while the other two cases correspond to a discrete release of a pollutant. Case (b) might portray a "first flush" type of scenario, while Case (c) might describe a delayed mobilization of a pollutant or a contribution from a relatively remote part of a catchment. In all cases, the initial concentration of solute in the pond was specified as zero. A total of eighteen simulations were carried out covering all the combinations of pipe diameter, initial water level and pollutograph. Following some initial sensitivity tests, a time step of 0.025 hours was used in all the simulations.

3. RESULTS AND DISCUSSION

Outflows and water levels

Figures 3 and 4 show the temporal variation of outflow and water level, respectively, for the six pond configurations considered, comprising three submerged pipe diameters and two initial water level conditions. Also shown on the figures is the inflow hydrograph. The outflows shown are the total outflows through both outlet devices. Figure 3 shows that greater attenuation of the peak outflow occurs when the pond is initially empty than when it is initially full, as would be expected, because in the former case water can be stored between the weir crest and the base of the pond. Note that in all cases when the pond is initially empty the outflow is initially zero, however, when the pond is initially full, outflow through the submerged pipe begins instantaneously at the start of the simulation. In the case of the largest pipe, the outflow decreases during the first hour of the simulation because the water level (see Figure 4) falls due to the pipe outflow exceeding the inflow. Once inflow exceeds pipe outflow, however, the water level rises and the pipe outflow follows suit. A similar type of behaviour occurs for the 0.1m diameter pipe (more clearly seen in the water level response), but the initial emptying of the pond is very short lived.

When the pond starts empty, good attenuation of the peak outflow occurs for all pipe diameter cases. In the case of the largest pipe diameter, outflow only occurs through the pipe because the water level never exceeds the weir crest elevation. For the 0.1m diameter case the water level exceeds the weir crest elevation after about 2 hours, which causes the rapid increase in total outflow shown in Figure 3. There is a corresponding sudden change in slope on the recession limb of the outflow hydrograph as the weir flow ceases. Subsequent outflow then occurs through the pipe only. When there is no pipe (corresponding to the 0.0m diameter case), outflow doesn't begin until about 1.9 hours, which is when the water level exceeds the weir crest. Interestingly, the 0.1m diameter pipe case provides the best peak outflow attenuation, but the pond takes about 16 hours to fully drain. With the 0.2m diameter pipe, the poorer peak outflow attenuation is traded against a more rapid draining of the pond. This trend of poorer attenuation but quicker draining will continue with further increases of pipe diameter. This can be an important consideration if a pond is subject to a sequence of inflow events, since an event will not be attenuated effectively if the preceding one is still occupying a significant part of the pond volume (Guo, 2002).

When the pond starts full, there is virtually no attenuation of the peak outflow except in the case of the largest pipe diameter. In this case, the initial draw down of the water level provides some storage volume, and the water level never exceeds the weir crest elevation, so all the outflow occurs through the submerged pipe. For the 0.1m pipe diameter case, outflow occurs through both pipe and weir, but once the water level falls below the weir crest (see Figure 4), outflow is through the pipe only. When there is no pipe (corresponding to the 0.0m diameter case), the water level cannot fall below the weir crest, so outflow only

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occurs through the weir, and the water level in the pond approaches the weir crest elevation asymptotically at some time after 16 hours. For the other two pipe cases, the pond empties after the inflow has stopped, much more quickly with the larger pipe diameter.

A final point of interest, which is more easily seen in the water level results (Figure 4), is that for the 0.0m and 0.1m diameter pipe cases, the response of the pond becomes independent of the initial water level condition once inflow has ceased. This is shown by the merging of the empty start and full start outflow hydrographs in each of these cases.

Solute concentrations

Figure 5 shows the temporal variation of solute concentration in the outflow for each of the three inflow solute concentration cases considered. For each case the figure shows outflow pollutographs from the six pond configurations together with the corresponding inflow pollutograph. For reasons of clarity the solute concentration axis for Cases (b) and (c) has been truncated, hence the maximum inflow concentration (100 mg/l) is not shown. In all cases, the outflow solute concentration rises to a peak and then falls. Once inflow has stopped (3.2 hours) the concentration remains constant. This happens because the pond is modelled under the assumption that it is well mixed. Hence, even though outflow continues, solute concentrations in the pond do not change, because under the well-mixed assumption the solute concentration in the pond water is the same as the concentration in the outflow water. The peak solute concentrations in the outflow are summarised in Table 1. These data show two clear trends: firstly, larger concentrations occur when the pond starts empty compared to it starting full, and secondly, in all cases concentrations increase with increasing submerged pipe diameters. Both of these are related to the volume of water in the pond. Solute concentrations are lower in cases where there is a greater volume of water available for diluting the pollutant. Hence, ponds that are initially full have a greater diluting potential than those that start empty, and since larger pipe diameters allow greater draw down of water levels (i.e. are associated with smaller volumes of water in the pond), they reduce the dilution capacity of the pond. Below, results from Case (b) are compared and contrasted against results from Case (c).

Table 1 (and Figure 5) shows that when the pond starts empty, pollutants that are carried into the pond during the rising limb of the inflow hydrograph are less well diluted than those that are carried into the pond during the falling limb. Indeed, there is a factor of about 2 between these (the ratios of the peak solute concentrations being 2.28, 2.22 & 1.89 for pipe diameters of 0.0m, 0.1m & 0.2m, respectively). Again, this is explained by considering the volume of water available for dilution. When the pond starts empty, Figure 4 shows that water levels are higher (i.e. pond water volumes are larger) after 2 hours (the mid-time of the pollutograph for Case (c)) than they are after 1 hour (the mid-time of the pollutograph for Case (b)). Hence dilution is higher and concentrations are lower for Case (c) compared to Case (b).

When the pond starts full, however, there is a much smaller difference between the two pollutograph cases (ratios of the peak concentrations being 0.85, 0.87 & 1.05 for pipe diameters of 0.0m, 0.1m & 0.2m, respectively), which reflects the smaller difference between the water levels (and volumes) at times of 1 and 2 hours shown in Figure 4 for the pond starting full case. Interestingly (and in contrast to the pond starting empty case), when the pond starts full the larger peak concentrations are found for the later occurring pollutograph (Case (c)), except for the largest pipe diameter. Since water levels are (slightly) higher for Case (c) than for Case (b), this suggests that water volume is not the only parameter that determines peak solute concentrations in the outflow, but it certainly appears to be the dominant one.

An analysis of the final solute concentrations (i.e. those at a time of 4 hours) yields similar trends to the peak concentrations, but there are a few anomalies for the largest pipe diameter. For example, for Case (b) the configuration of the largest pipe diameter and the pond starting empty gives the lowest final concentration.

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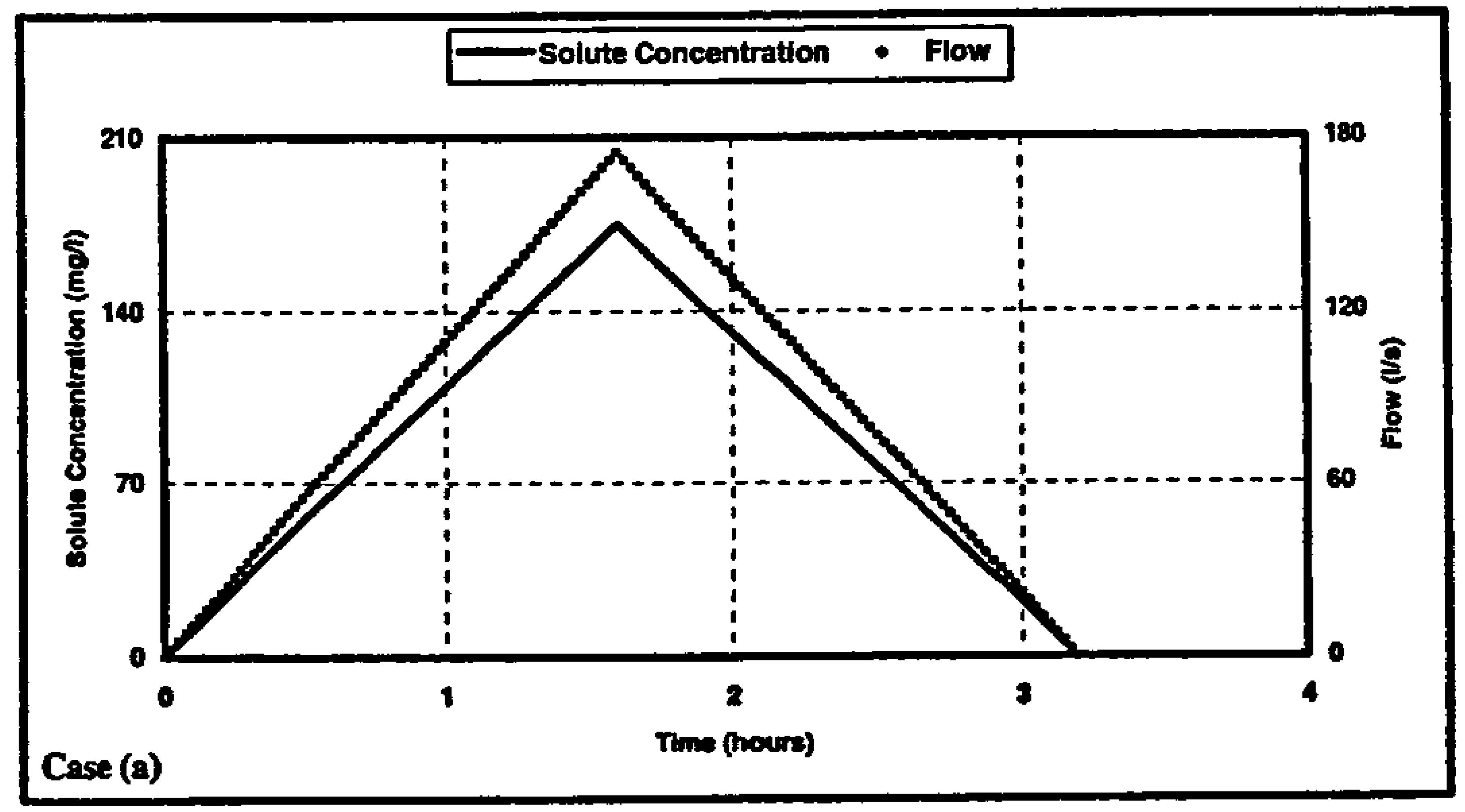
4. CONCLUSIONS

The main findings drawn from the simulations are:

1. Greater attenuation of the peak outflow occurs with increasing available storage in the pond.
2. Greater available storage can be achieved by increasing the diameter of the submerged pipe, because this enables a preceding storm to be more quickly drained. However, this adversely affects the peak flow attenuation because the full storage capacity of the pond is rarely used.
3. The period between storms is also an important parameter here.
4. The greater the volume of water in the pond when pollutants enter it, the greater is their dilution.
5. Greater volumes of water in the pond are promoted by decreasing the diameter of the submerged pipe.
6. Particularly when the pond starts empty, there is less dilution available during the rising limb of the inflow hydrograph than there is during the falling limb. Hence "first flush" pollution events are not diluted particularly well.
7. There are conflicts between achieving good peak flow attenuation and achieving high dilution of pollutants.
8. Further work is required to understand how pond designs can be improved to address these conflicts. In this, consideration should be given to the role of the inter-event period and to the behaviour of suspended pollutants as well as dissolved ones.

Figures



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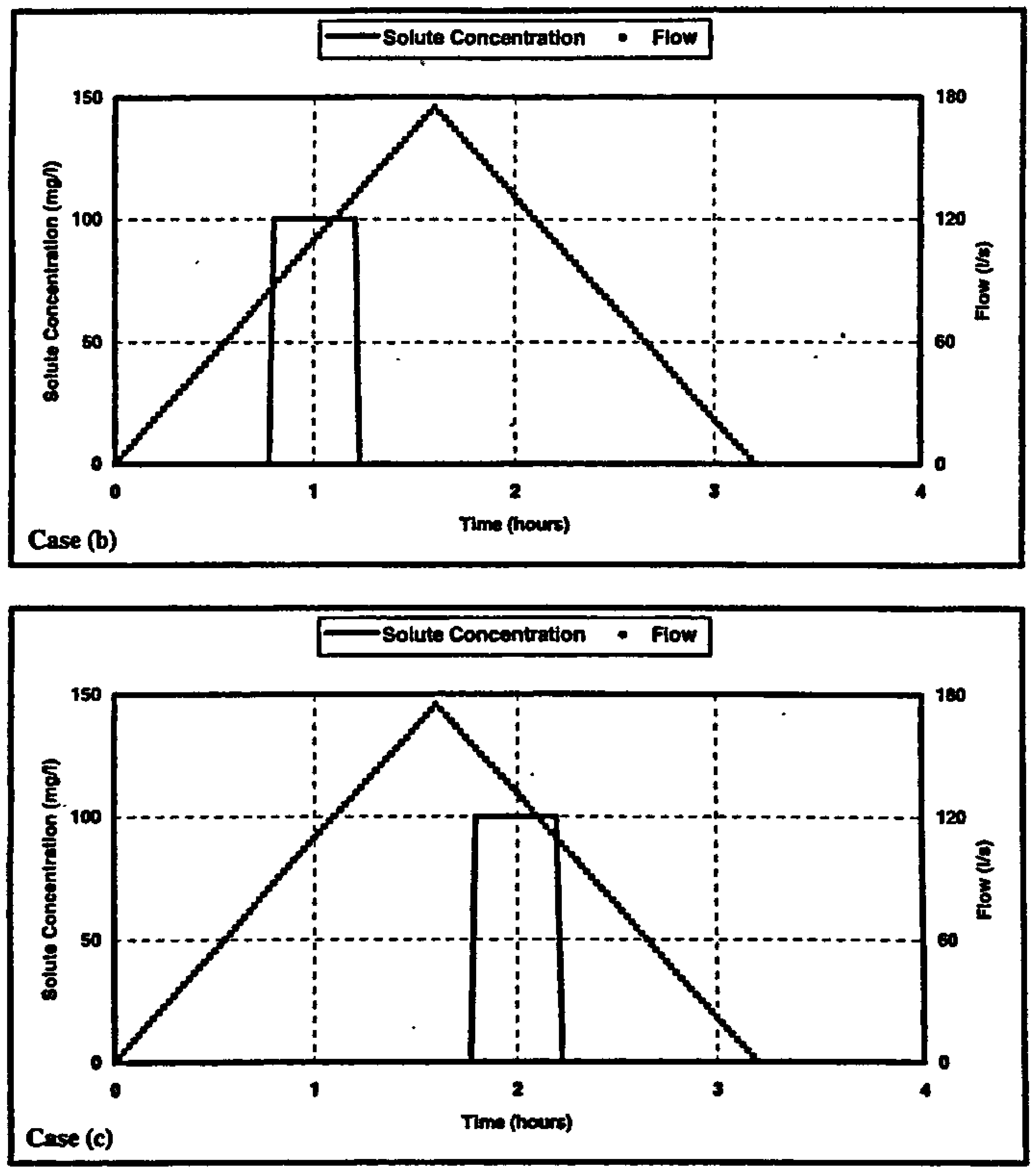

Figure 2: Inflow hydrograph and pollutographs used in the simulation.

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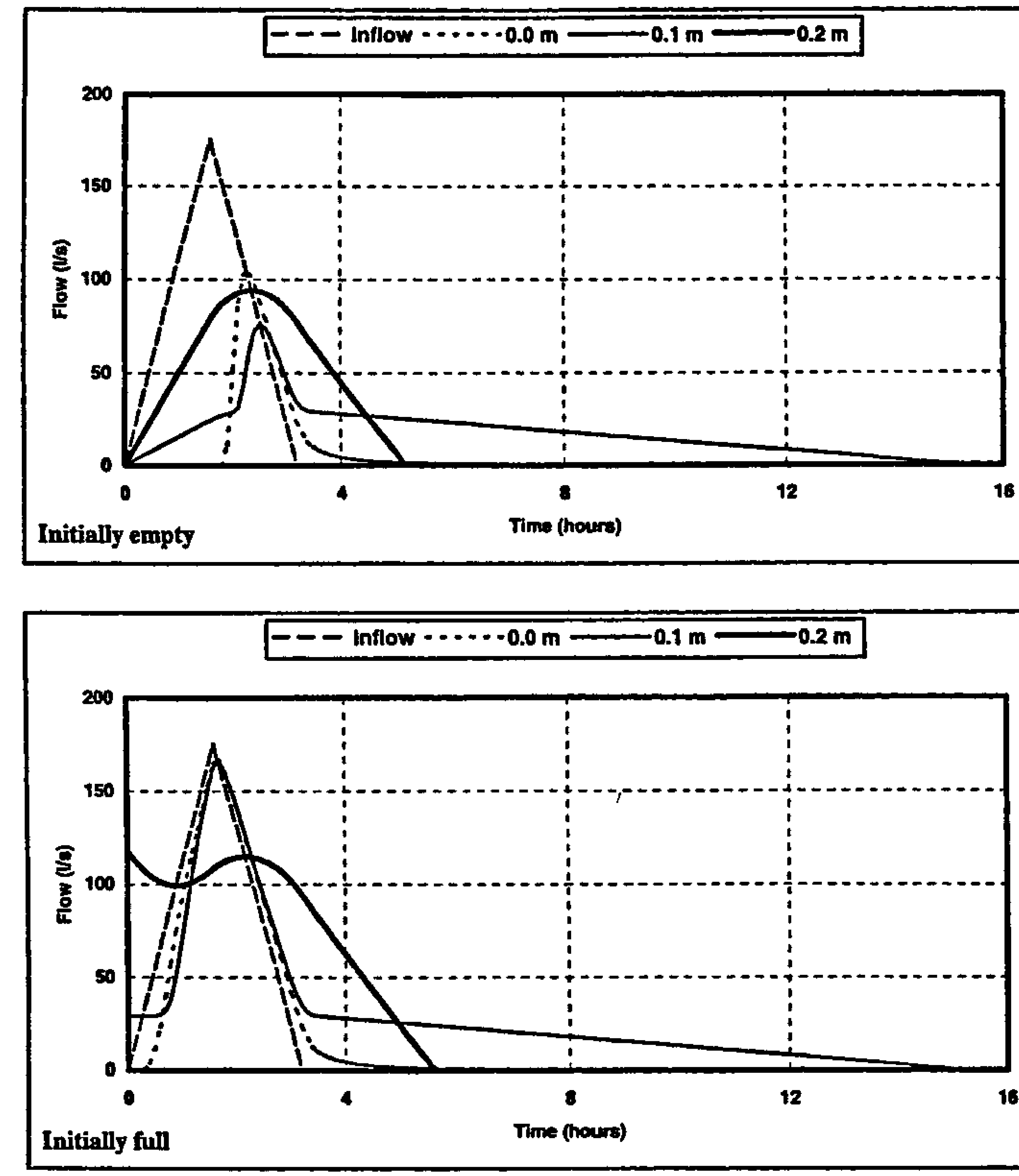


Figure 3: Inflow and outflow hydrographs for different pipe diameters and initial conditions.

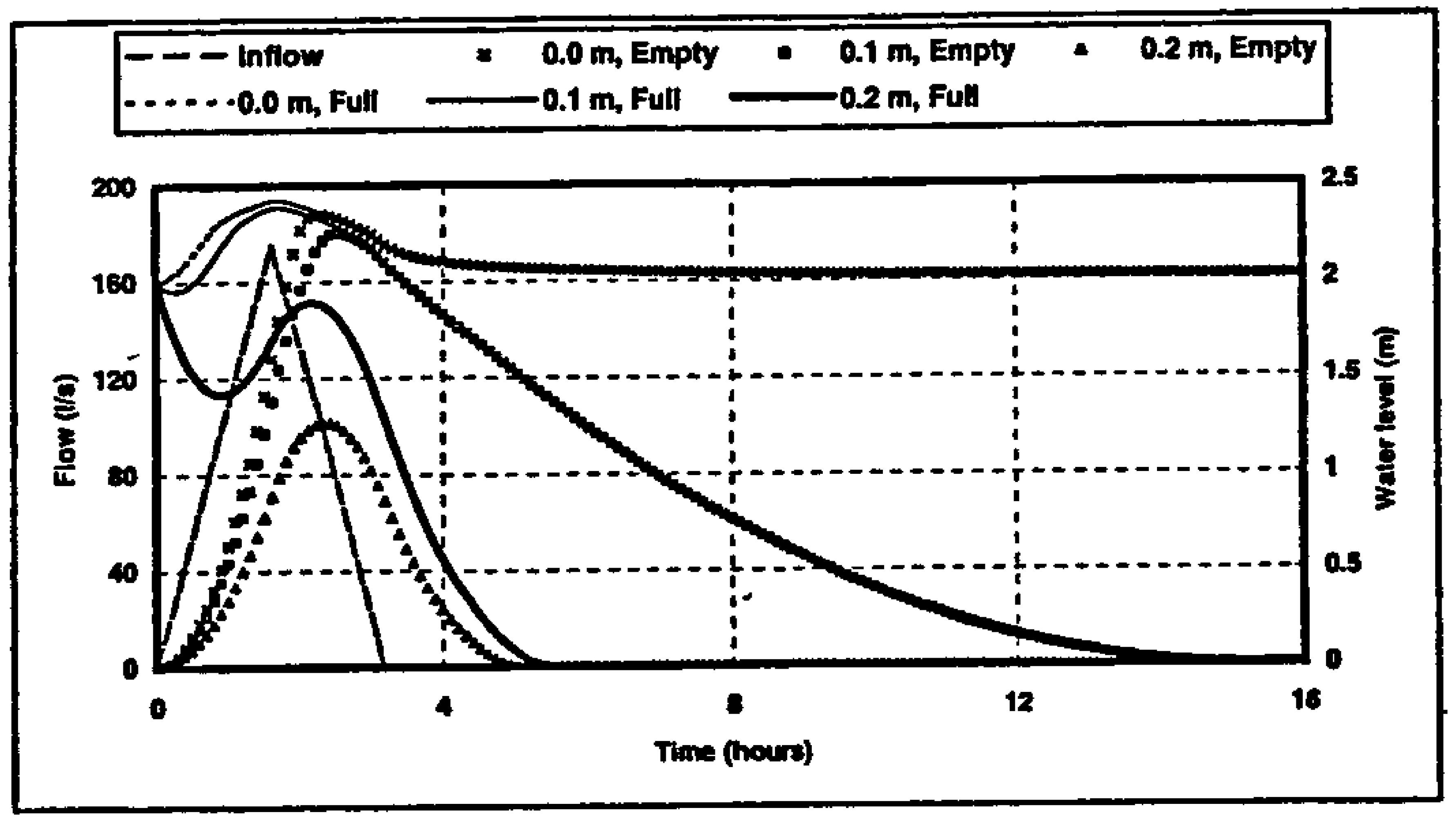
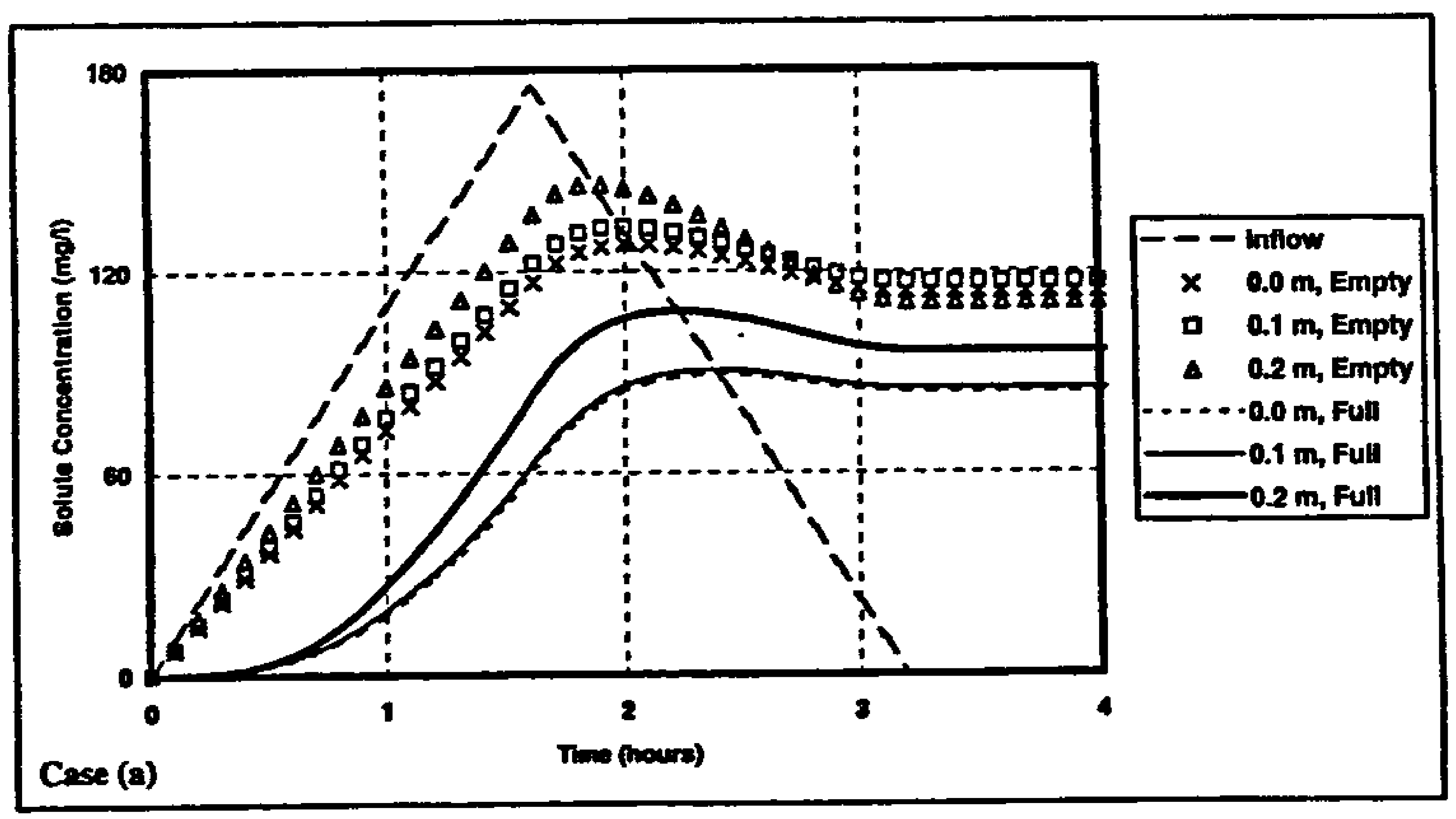


Figure 4: Inflow hydrograph and water levels for different pipe diameters and initial conditions.



Case (a)

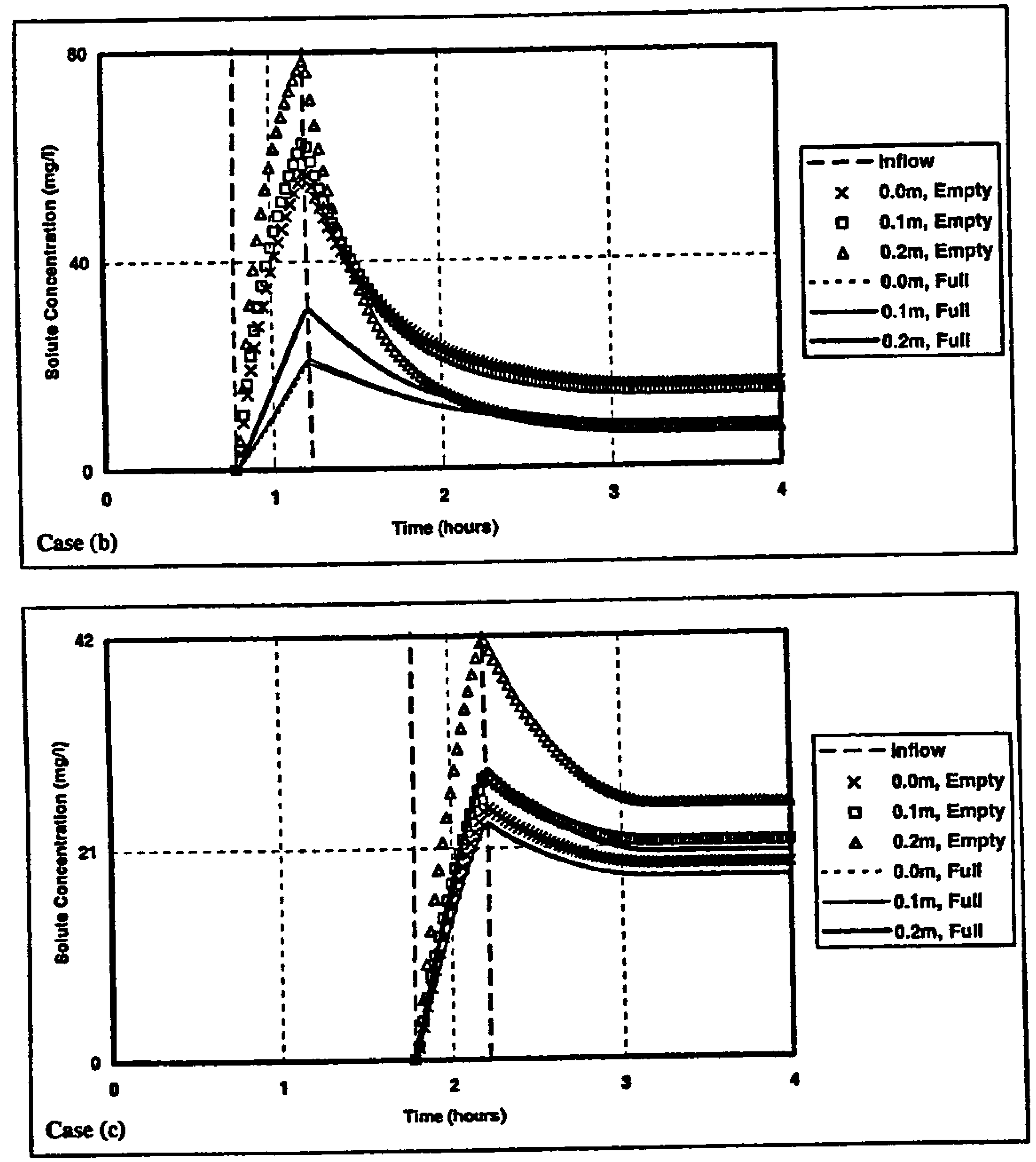


Figure 5: Inflow and outflow pollutographs for different pipe diameters, initial conditions and inflow pollutographs: Case (a) continuous release during inflow; Case (b) discrete release during rising limb of inflow; Case (c) discrete release during falling limb of inflow.

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Table 1: Peak solute concentrations (mg/l) in the outflow for three inflow pollutograph cases and six pond configurations.

Pipe diameter (m)	Pollutograph Case (a)		Pollutograph Case (b)		Pollutograph Case (c)	
	Initially empty	Initially full	Initially empty	Initially full	Initially empty	Initially full
0.0	128.1	89.4	56.9	19.8	25.0	23.2
0.1	133.3	90.6	62.5	20.6	28.1	23.6
0.2	146.1	108.0	78.3	30.2	41.5	28.8

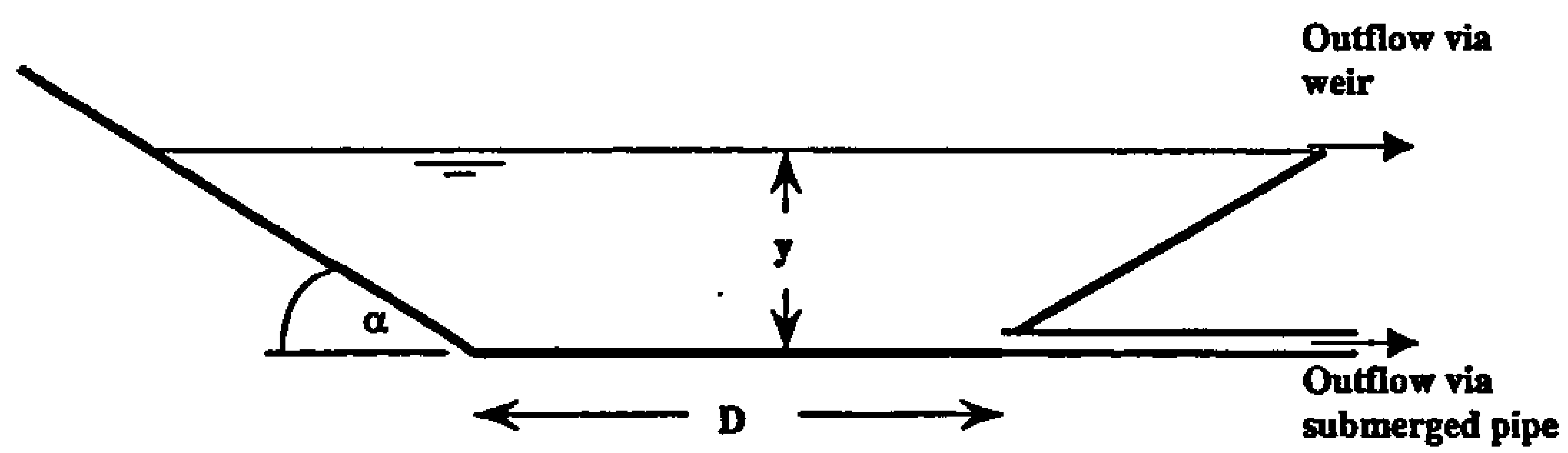

Figure 1 Section through generic, conical detention pond showing high water level conditions for which outflow occurs through both submerged pipe and weir (symbols are defined in the text).

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Using mathematical modelling to inform on the ability of stormwater ponds to improve the water quality of urban runoff

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Abstract This paper concerns the mathematical modelling of flow and solute transport through stormwater ponds. The model is based on appropriate lumped system conservation equations that are solved using standard numerical techniques. The model was used to route a first flush pollution scenario through a cylindrical pond for 16 combinations of elevation and diameter of a submerged pipe outlet, in conjunction with a high level weir. Higher pipe elevations and smaller pipe diameters created larger pond volumes and hence led to greater dilution of the pollutant. In contrast, lower pipe elevations created larger storage volumes, leading to better flow attenuation. Interestingly, larger pipe diameters improved peak flow attenuation, even though the storage used decreased.

Keywords Dilution; flow attenuation; mathematical modelling; solute transport; stormwater ponds

Introduction

Stormwater ponds are frequently used to attenuate flows derived from surface water drainage and, depending on their design, they provide a greater or lesser degree of water quality treatment. Detention basins (or dry ponds) store stormwater for relatively short periods of time and provide little water quality treatment, whereas retention ponds (or wet ponds) store stormwater for much longer, which enables most of the suspended material to settle out, thus significantly reducing the turbidity of the outflow. Since much of the pollutant load is bound to the sediment (Ellis and Hvitved-Jacobsen, 1996), the removal of the suspended sediment further reduces the impact on the receiving watercourse. In addition, a retention pond's permanent pool of water provides the sort of conditions that allows pollutants to degrade or to become more securely bound to plant or mineral substrates. However, some pollutants exist in solution (and do not settle out) and some sediment particles may be so small that they pass through the retention pond before settling, taking any bound pollutants with them. The latter is particularly likely to happen when the inflow to, and the outflow from, the pond are both high, since under these conditions short-circuiting is more likely to occur, caused by the development of a momentum driven preferential flow path through the pond.

To date, little attention has been devoted to the fate of solutes and fine sediments in retention ponds, and little is known about: (a) the degree of physical dilution that is achieved by the mixing of the inflow with the water in the permanent pool; and (b) how sensitive the dilution is to the outlet configuration. The aim of the work described in this paper is to address these issues by undertaking simulations of solute transport through a simplified retention pond. The simulations were carried out using a mathematical model of a retention pond that was developed by the study team.

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Model development

The mathematical model consists of two components: a flow model and a solute transport model; and in both cases the pond is modelled as a deterministic, lumped system. In the work described in this paper the pond was assumed to be cylindrical, having a single inlet and a dual outlet system.

Flow model

Flow through the pond is modelled using a standard storage routing method (Shaw, 1994; Mays, 2001) that is based on the following equation that describes the conservation of volume of water:

$$\frac{dV}{dt} = Q_i - Q_o \qquad (1)$$

where V is the volume of water in the pond (often termed storage), Q_i is the volumetric flow rate of water entering the pond (inflow) and Q_o is the volumetric flow rate of water leaving the pond (outflow). Equation (1) is solved to give the temporal variation (hydrograph) of outflow, assuming that the inflow hydrograph is known. The outflow passes through two outlet devices that are described by hydraulic head-discharge equations. Thus the outflows are actually calculated from predicted water levels in the pond. The outlet devices considered here are: a submerged horizontal pipe (modelled as a submerged orifice) located at various elevations above the base of the pond and a v-notch weir located at a fixed elevation above the base of the pond. The hydraulic equations for these two devices are (Chadwick and Morfett, 1998):

(a) submerged pipe

$$Q_{sp} = aC_D\sqrt{2gH} = K_{sp}H^{0.5} \qquad (2)$$

where Q_{sp} is the flow through the pipe, a is the cross-sectional area of the pipe, C_D is the coefficient of discharge, g is the acceleration due to gravity, H is the head and K_{sp} is a constant:

(b) v-notch weir

$$Q_{vnw} = \frac{8}{15}C_D\sqrt{2g}\tan(\gamma/2)H^{2.5} = K_{vnw}H^{2.5} \qquad (3)$$

where Q_{vnw} is the flow through the weir, γ is the weir angle, K_{vnw} is a constant and the other symbols are as previously defined.

The head for the submerged pipe is the difference between the water level in the pond, y, and the pipe elevation, y_{sp}; the head for the v-notch weir is the difference between the water level in the pond and the weir crest elevation, y_{vnw}. Clearly, outflow through the pipe occurs whenever $y > y_{sp}$, and outflow through the weir occurs whenever $y > y_{vnw}$. The total outflow, Q_o, is simply the sum of the pipe and weir outflows. Modelling the pipe outlet using equation (2) implies certain assumptions and simplifications that may not be true at all times in practice (Urbonas and Stahre, 1993; Mays, 2001). Nevertheless, equation (2) has the attraction of simplicity.

Equation (1) can be solved in several ways. Since inflow hydrographs can rarely be represented by a simple function, a numerical approach is usually adopted. Traditionally, hydrologists have employed the storage-indication method (Shaw, 1994; Mays, 2001) although there are many standard numerical algorithms for ordinary differential equations that could be used instead (Mason and Stocks, 1987; Quinney, 1987; Griffiths and Smith, 1991; Chapra and Canale, 1998). The latter route is followed here mainly because these methods can also be used for the solute transport model.

Expressing equation (1) in a standard time-weighted finite difference form gives:

$$\frac{V^{n+1} - V^n}{\Delta t} = \theta(Q_i - Q_o)^{n+1} + (1-\theta)(Q_i - Q_o)^n \quad (4)$$

where superscripts n + 1 and n refer to values evaluated at two times separated by the time step, Δt, and θ is a time weighting parameter ($0 \leq \theta \leq 1$). It can be assumed that any term evaluated at time n is known, so that only terms evaluated at time n + 1 need to be calculated. The left-hand side of equation (4) represents the change in volume that occurs during the time step. This can be expressed in terms of the surface area of the pond, A, and the water level, y, as follows:

$$V^{n+1} - V^n = \tilde{A}(y^{n+1} - y^n) \quad (5)$$

where the ~ overbar indicates an average value during the time step. Combining equations (4) and (5) and re-arranging gives:

$$y^{n+1} = B - \frac{\Delta t\theta}{\tilde{A}} Q_o^{n+1} \quad (6)$$

where:

$$B = y^n + \frac{\Delta t(1-\theta)}{\tilde{A}}(Q_i - Q_o)^n + \frac{\Delta t\theta}{\tilde{A}} Q_i^{n+1} \quad (7)$$

All the terms on the right-hand side of equation (7) are known, either because they are all evaluated at time n or, in the case of the third term, because all values of the inflow are known. Replacing the unknown outflow in equation (6) by the sum of equations (2) and (3) and re-expressing the outlet heads gives:

$$y^{n+1} = B - \frac{\Delta t\theta}{\tilde{A}}\left(K_{sp}(y^{n+1} - y_{sp})^{0.5} + K_{vnw}(y^{n+1} - y_{vnw})^{2.5}\right) \quad (8)$$

Generally, equation (8) is non-linear in y^{n+1} and needs to be solved via iteration. When $\theta = 0$, however, a direct or explicit solution is possible. This corresponds to the Euler algorithm (Mason and Stocks, 1987; Quinney, 1987; Griffiths and Smith, 1991; Chapra and Canale, 1998). It is well known that the simplicity of this algorithm needs to be balanced against its relatively poor accuracy and potential for instability. Taking ($0 < \theta \leq 1$), however, though incurring extra computational cost, yields a more robust solution. Following standard practice, $\theta = 0.5$ (corresponding to the trapezium method (Quinney, 1987; Griffiths and Smith, 1991) or the Crank–Nicolson method (Griffiths and Smith, 1991)) was adopted. This gives a theoretically more accurate solution than other values of θ, and the algorithm has good stability characteristics.

Again, following standard practice, Newton–Raphson iteration (Griffiths and Smith, 1991; Chapra and Canale, 1998) was used to cater for the non-linear nature of equation (8). To simplify the detail of the iteration, the surface area used in equations (7) and (8) was taken as the area corresponding to y^n. To maintain the highest accuracy, this surface area could, instead, have been evaluated as the average of the areas corresponding to y^n and y^{n+1}, however, such a refinement was not considered to be necessary because it was anticipated that relatively small time steps would be used in the simulations.

Solute transport model

The solute transport model is based on a similar equation to the flow model, but in this case the equation describes the conservation of mass of solute:

$$\frac{d(VC)}{dt} = Q_iC_i - Q_oC_o \quad (9)$$

where C is the concentration of the solute in the pond, C_i is the solute concentration in the inflow and C_o is the solute concentration in the outflow. Equation (9) is solved to give the temporal variation of C_o, assuming that volumes and flows are known from the flow model and that the solute concentration in the inflow is known. Using a similar finite difference form, as before, gives:

$$\frac{(VC)^{n+1} - (VC)^n}{\Delta t} = \theta(Q_iC_i - Q_oC_o)^{n+1} + (1-\theta)(Q_iC_i - Q_oC_o)^n \quad (10)$$

Assuming that the pond is well mixed, so that solute that enters the pond is instantaneously and uniformly mixed throughout the water in the pond, then C_o can be replaced by C. Thus the pond is assumed to behave as a continuously stirred tank reactor (Chapra, 1997; Mihelcic, 1999). After some re-arrangement, equation (10) becomes:

$$C^{n+1}(V + \theta\Delta tQ_o)^{n+1} = C^n(V - (1-\theta)\Delta tQ_o)^n + \theta\Delta t(Q_iC_i)^{n+1} + (1-\theta)\Delta t(Q_iC_i)^n \quad (11)$$

Hence, C^{n+1} is found easily since all the other terms are known and there are no non-linearities. As before, θ was taken as 0.5, to enhance the accuracy of the calculation.

Simulations

The simulations were designed to identify some of the major influences on solute concentration in the outflows from retention ponds. The (cylindrical) geometry of the pond, the location and characteristics of the v-notch weir and the inflow hydrograph remained fixed whilst the elevation of the submerged pipe and its diameter were varied. Several, different temporal distributions of solute (pollutographs) in the inflow were considered, but here interest focuses on just one.

The pond radius was specified as 10 m. The crest of the weir was located 2 m above the base of the pond, the weir angle was specified as 90° and the coefficient of discharge was specified as 0.6 (Chadwick and Morfett, 1998). The pipe was located at one of four elevations above the base of the pond (0 m, 0.5 m, 1.0 m, 1.5 m) and for each elevation four pipe diameters were considered (0.05 m, 0.1 m, 0.15 m and 0.2 m). The pipe's coefficient of discharge was specified as 0.6 (Chadwick and Morfett, 1998). The inflow hydrograph was an isosceles triangle of duration 3.2 hours and peak flow of 175 l/s. Thus the inflow volume was 1008 m^3, compared to a maximum available storage volume (below the weir crest) of 628 m^3. For each simulation the initial water level was set equal to the elevation of the submerged pipe. Note that the case where the pipe is located at the base of the pond corresponds to a detention basin, while the other three cases correspond to retention ponds.

The pollutograph considered here consisted of a short pulse of solute occurring during the rising limb of the inflow hydrograph and portrays a "first flush" type of scenario. In all cases, the initial concentration of solute in the pond was specified as zero. A total of 16 simulations were carried out covering all the combinations of pipe elevation and diameter. Following some initial sensitivity tests, a time step of 0.025 h was used in all the simulations.

Results and discussion

The main characteristics of the flow attenuation and water quality treatment shown by the simulations are described in the following sections. Attention focuses on the peak outflow and the peak solute concentration in the outflow. The conservation of flow volume and solute mass was checked for all the simulations and was found to be excellent.

Flow attenuation

A typical example of the flow and water level variations from the simulations is shown in Figure 1. Three outflows are shown: the outflow through the pipe, the outflow through the weir and the total outflow. Following the start of inflow, outflow initially occurs only through the pipe until the water level reaches the weir crest. Thereafter, outflow occurs through both outlets until, on the recession, the water level falls below the weir crest, after which outflow only occurs through the pipe.

The peak outflow is directly related to the maximum water level because both outlet devices are driven by their hydraulic heads. Generally, the maximum water level achieved depends on the pipe elevation, the pipe diameter and the inflow volume, however, since the latter was kept constant here, the combination of the former two controlled the flow attenuation achieved.

An interesting way of showing all the results together is given in Figure 2, in which the peak outflow is plotted against its time of occurrence. Since in all cases the peak outflow should occur when the outflow hydrograph crosses the recession limb of the inflow hydrograph, all the data should fall on the inflow recession limb. Figure 2 shows that this is indeed the case, and indicates that there are no gross errors in the simulations. The spread of the data shows the range of peak flow attenuation achieved over the sixteen combinations of pipe elevation and diameter. The data is grouped according to pipe elevation, showing that, in broad terms, attenuation improves (peak outflow decreases) with decreasing pipe elevation. This reflects the greater storage volume that is available when the pipe outlet is located closer to the base of the pond.

Figure 3 shows the same data in a more detailed way, and exposes the relationship between peak flow attenuation, storage, pipe elevation and pipe diameter. More specifically the figure shows the peak flow ratio (defined as the ratio of peak outflow to peak inflow) plotted against the maximum storage used (defined as (maximum water level reached − initial water level) × surface area of pond) for all the simulations. This shows that for the same pipe elevation, increasing attenuation is found with increasing

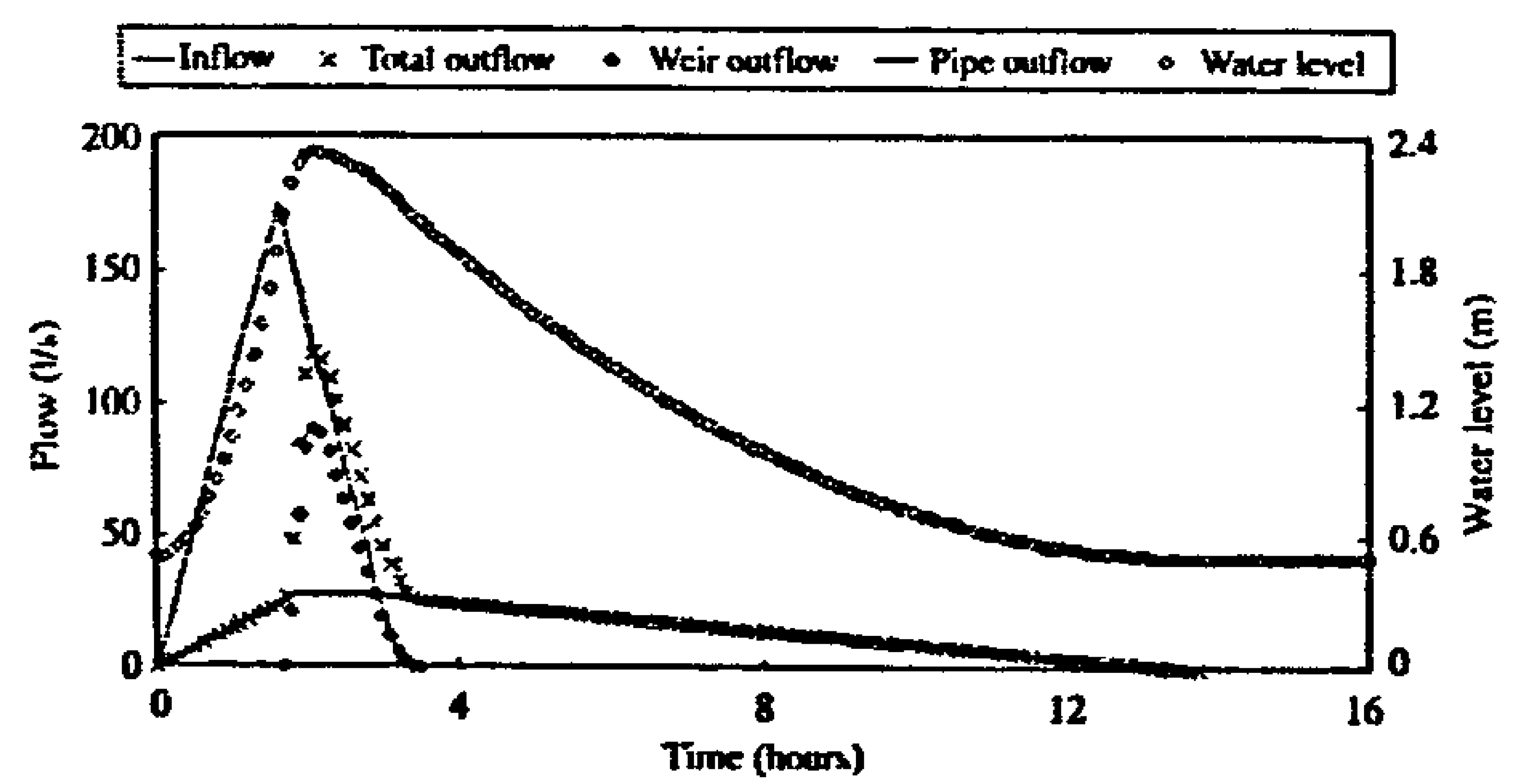

Figure 1 Flows and water level for 0.1 m diameter pipe located 0.5 m above base of pond

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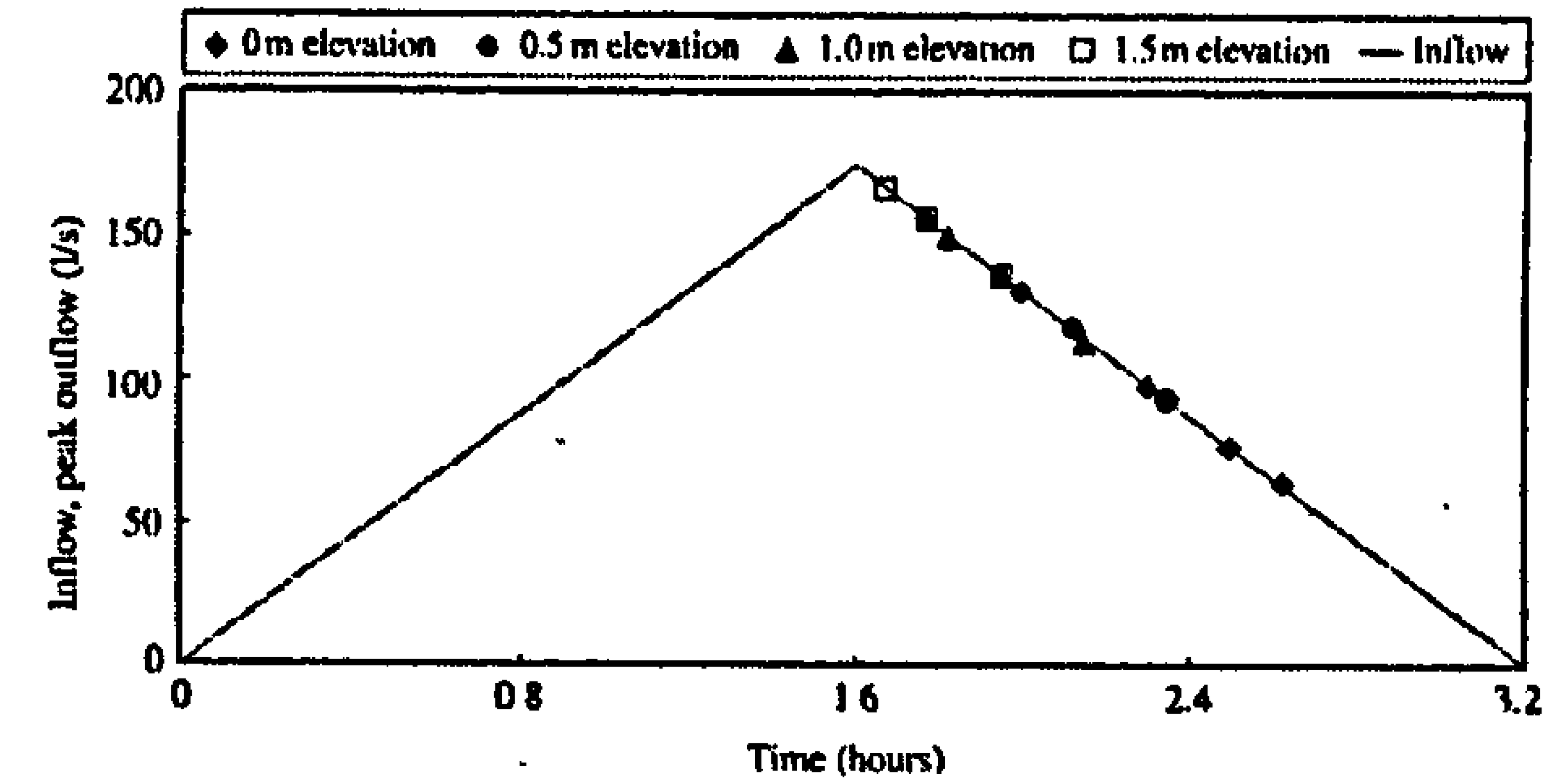

Figure 2 Peak flow attenuation for all simulations plotted for different pipe elevations

pipe diameter, but the storage used decreases. This reflects the lower maximum water levels achieved (because with a larger diameter pipe more water is discharged from the pond and a smaller volume is stored). Indeed, when the 0.2 m pipe is located at 0 m and 0.5 m above the base of the pond the water level does not reach the weir crest because of the large capacity of the pipe outlet. These two cases then provide identical flow attenuation (using the same storage volume), and are to some extent anomalous.

Water quality treatment

A typical example of the pollutographs and pond volume variations from the simulations is shown in Figure 4. Once pollutant enters the pond it immediately appears in the outflow because the pond is assumed to be completely mixed. The outflow concentration then rises and reaches a peak at the cessation of the pollutant inflow, following which it gradually reduces to a constant level. This recession phase is caused by dilution of the pollutant in the pond by the further inflow of clean water. Once the inflow stops, the pollutant concentration in the outflow remains constant until eventually the outflow ceases, after which the outflow concentration is set to zero.

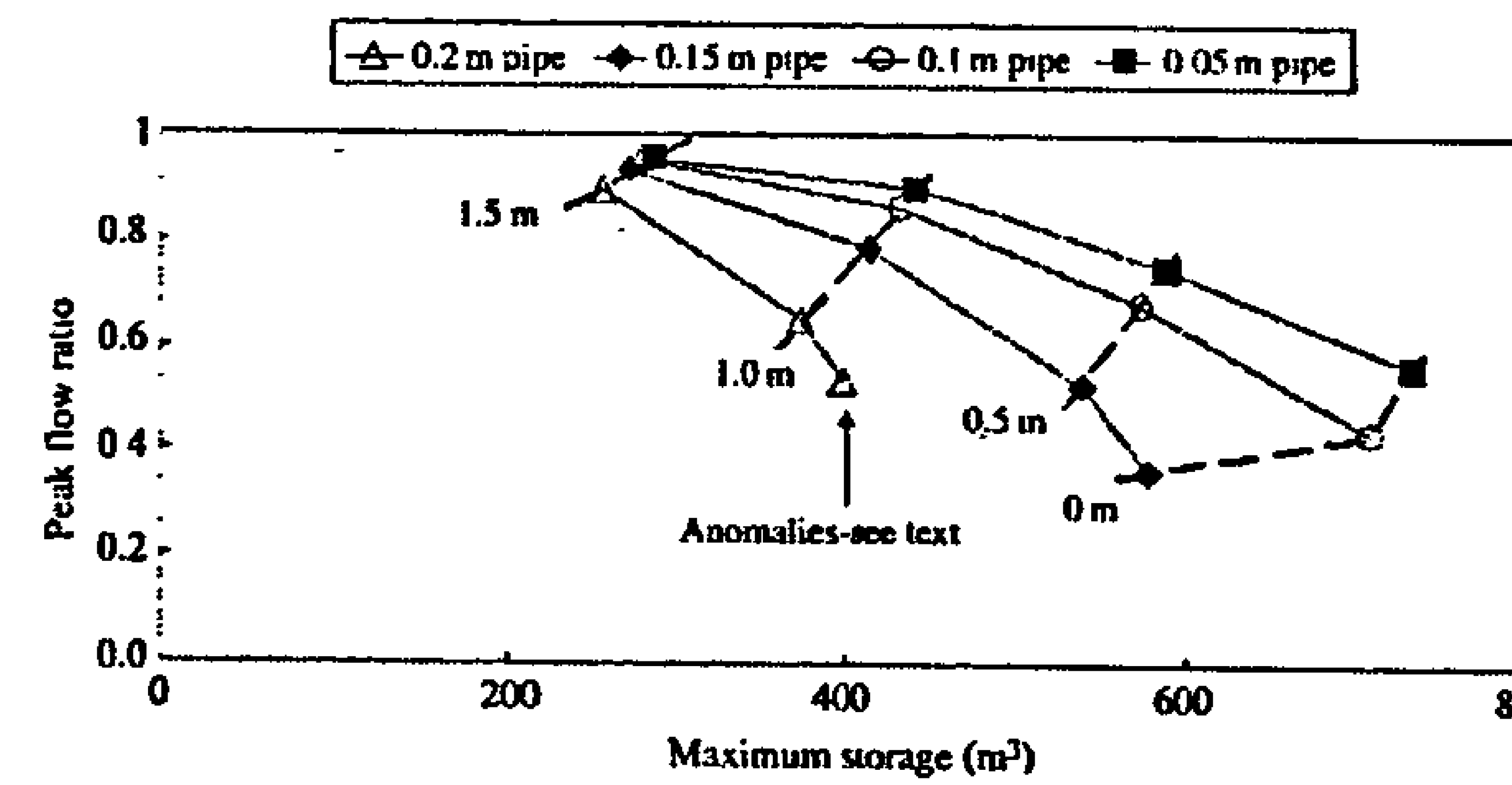

Figure 3 Relationship between peak flow attenuation, storage and outlet pipe configuration; the legend shows pipe diameters and the dashed lines indicate pipe elevation

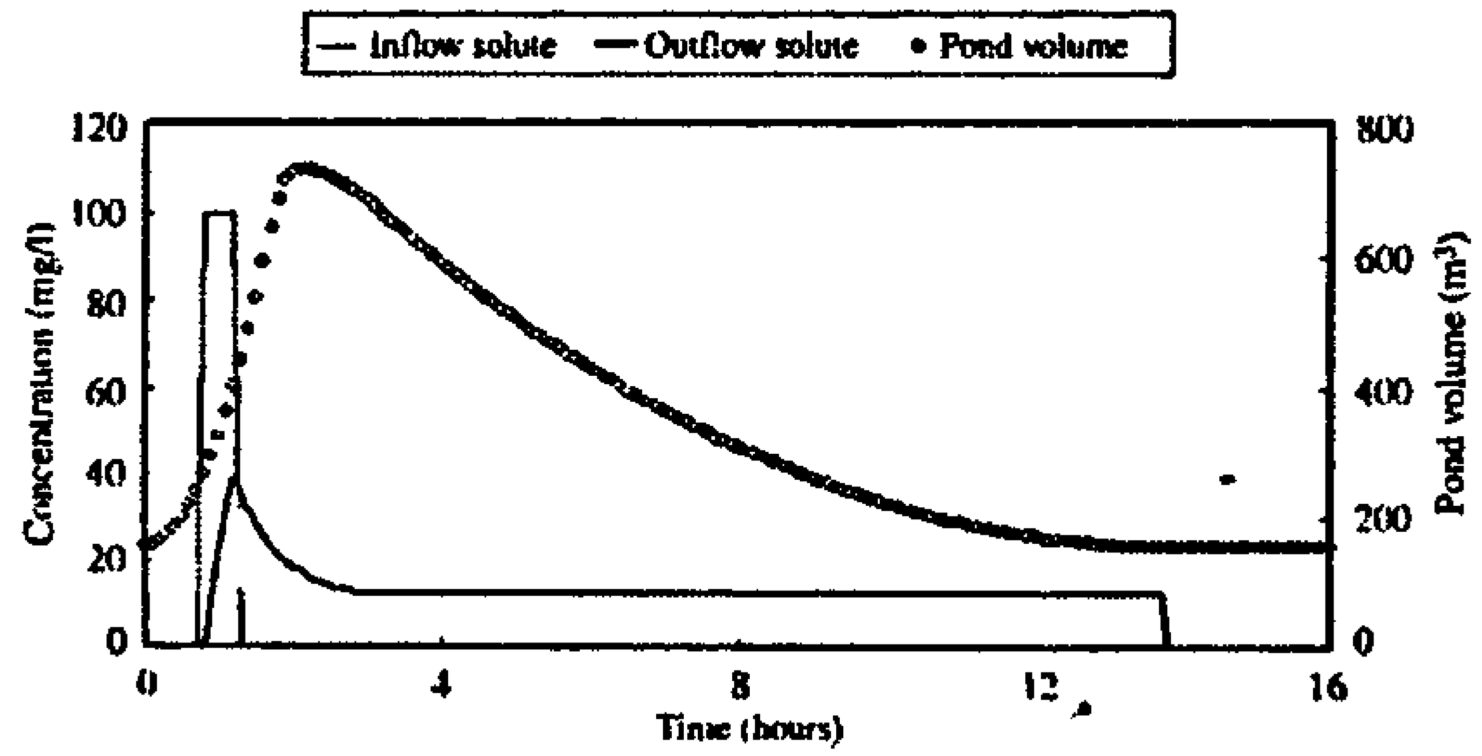

Figure 4 Solute concentration and pond volume for the same case shown in Figure 1 (0.1 m diameter pipe located 0.5 m above base of pond)

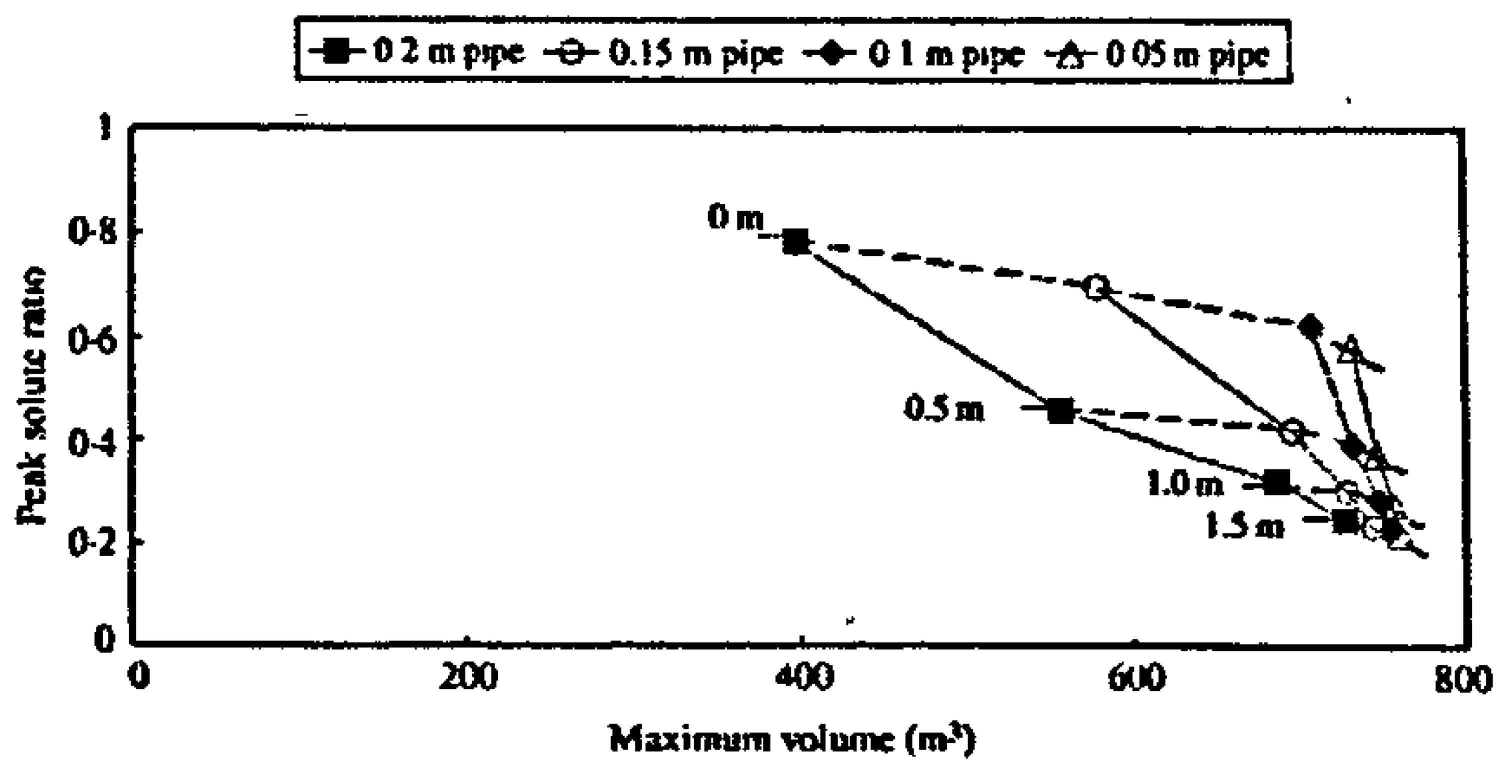

Figure 5 Relationship between peak solute ratio, storage and outlet pipe configuration; the legend shows pipe diameters and the dashed lines indicate pipe elevation

The relationship between peak outflow concentration, storage, pipe elevation and pipe diameter is shown in Figure 5, which shows the peak solute ratio (defined as the ratio between peak outflow solute concentration and peak inflow solute concentration) plotted against the maximum pond volume (defined as maximum depth achieved × surface area of pond). This is a more appropriate measure to use here than the maximum storage used in Figure 3 because the pollutant is mixed within the whole volume of the pond (including the permanent pool below the elevation of the pipe). Clearly, higher dilution of the incoming pollutant is achieved when there is a larger volume of water in the pond, i.e. better dilution is found when the pipe outlet is located further above the base. The figure also shows that increased dilution is found when the pipe diameter decreases. This happens because the volume of water in the pond tends to be greater when the pipe outlet capacity is reduced. There is a greater sensitivity to pipe diameter when the pipe is at a lower elevation.

Conclusions

Simulations of flow through cylindrical retention ponds having a submerged pipe outlet and a higher level weir outlet were generally consistent with the conventional idea that flow attenuation improves as available storage increases. Peak flow attenuations were in

the range 0.36–0.96. Interestingly, by increasing the diameter of the pipe outlet, peak flow attenuation was improved even though a reduced storage volume was used. Simulations of the dilution of a dissolved first flush pollutant load resulted in the ratio of the peak concentration in the outflow to the peak concentration in the inflow being in the range 0.2–0.8. Larger dilutions corresponded with larger pond volumes, created either by a higher elevation, or by a smaller diameter, of the submerged pipe. There is an interesting conflict between achieving good dilution and good flow attenuation because the former requires a large percentage of the pond volume to be occupied by water at the start of the inflow, whereas the latter requires a small percentage of the pond volume to be initially occupied by water.

Acknowledgements

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Assessing the effects of design and climate change on sediment removal in urban stormwater ponds

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Abstract Urban stormwater pond design normally only considers single storm events, does not explicitly consider climate change and is often inconsistent, with some approaches emphasising flow attenuation and others emphasising water quality enhancement. These design issues were explored for sediment removal (by settling) through modelling generic cylindrical ponds sized using current UK guidance. Results showed that ponds designed for flow attenuation had a higher sediment removal efficiency than those designed for water quality enhancement (78% *vs* 21% removal of incoming sediment, respectively, for the 1 in 2 year storm event). Sediment removal efficiency remained almost unchanged when multiple rather than single storm events were routed through ponds, but decreased with increasing storm event magnitude. Overall, decreased sediment removal is likely from the more frequent and intense storm events predicted due to climate change. Urban stormwater ponds designed for flow attenuation are more successful for both flow and sediment attenuation.

Key words climate change; design; modelling; ponds; retention basins; Scotland; sediment; SUDS; urban stormwater

INTRODUCTION

During the last decade stormwater ponds have been increasingly used worldwide to minimise the impact of urbanisation on the water environment. In these systems (also known as retention ponds or basins in the UK), which are normally defined as containing a permanent pool of water, flow attenuation occurs by temporary storage of runoff followed by a delayed and slow release to the receiving watercourse. Water quality improvement occurs primarily by the capture of sediment in the pond, through the settling of suspended solids, since many of the major pollutants carried by surface runoff are attached to sediment particles. Sediment removal is highly dependent on pond design, particularly the characteristics of the permanent pool of water. A larger permanent pool enhances settling by increasing residence time, as well as providing habitat for aquatic vegetation, which enhances filtration and pollutant removal by nutrient uptake and microbial degradation.

Despite the widespread deployment of urban stormwater ponds there is still considerable variability in the guidance for pond design. Some approaches stress designing for flow attenuation, e.g. in the UK, the Construction Industry Research and

Information Association) (CIRIA, 1993) recommended that ponds should be sized to attenuate the 1 in 25 and 1 in 100 year flood events, whilst other approaches emphasise design for water quality enhancement. In the UK, the normal approach for pond design for water quality enhancement is to size the permanent pool to hold one or more treatment volumes (*Vt*), defined as the volume of runoff generated by the first 12–15 mm of rainfall on the impervious catchment surface, thereby capturing the first flush of storm runoff which is typically the most polluted. Recent research suggests that one *Vt* may be an acceptable size criterion for ponds for most low risk urban sites (McLean *et al.*, 2005), smaller than the three to four *Vt* previously recommended (CIRIA, 2000).

The aims of this research were to investigate the effects on sediment removal efficiency of three aspects of current design methodology for urban stormwater ponds. Firstly, the different design approaches (flow attenuation and water quality enhancement); secondly, single *vs* sequential storm events (since normally only single storm events are considered, whereas, in reality, events often occur in sequence); and thirdly whether ponds designed using the current recommended methodology will provide adequate water quality improvement for the increased frequency and magnitude of storm events predicted as a result of climate change. The impacts of these design issues were explored through simulations of sediment capture by settling in generic cylindrical ponds.

MODELLING APPROACH

The mathematical model consisted of two components: a flow model and a sediment transport model; and in both cases the pond was modelled as a deterministic, lumped system. The pond was assumed to be cylindrical, having a single inlet and a single outlet device.

Flow model

Flow through the pond was modelled using a standard storage routing method (Mays, 2001) that is based on the following conservation of water volume equation:

$$\frac{dV}{dt} = Q_i - Q_o \qquad (1)$$

where V is the volume of water in the pond, Q_i is the volumetric inflow rate of water, Q_o is the volumetric outflow rate of water and t is time. Equation (1) was solved to give the outflow hydrograph, assuming that the inflow hydrograph was known. Outflow from the pond occurred through a v-notch weir, and was calculated using a standard head-discharge equation (Chadwick & Morfett, 1998). Equation (1) was solved numerically using a standard time-weighted finite difference method (Griffiths & Smith, 1991) to give a nonlinear approximation, which was solved in a standard way using Newton-Raphson iteration (Chapra & Canale, 1998).

Sediment transport model

The sediment transport model computes the temporal distribution of suspended sediment concentration in the pond outflow (outflow sedigraph) for any specified distribution of suspended sediment in the inflow (inflow sedigraph). The model is based on a mass balance of sediment and caters for several transport mechanisms. It recognises that the concentration of suspended sediment in a stormwater pond (particularly during an inflow event) may not be uniform and that several flow-related processes, such as short-circuiting and flow-dependent settling, are likely to occur. Where short-circuiting occurs, the sediment has little opportunity to mix with the rest of the water in the pond and to settle. Whether it settles or not depends on the flow conditions and on the sediment characteristics. For example, heavy sediment particles are likely to settle under all flow conditions but light ones will only settle when the flow is weak, being carried out of the pond in the outflow otherwise. Elsewhere in the pond, the water is essentially static and suspended sediment in these areas settles under quiescent conditions. The above features were catered for by dividing the pond into two (lumped) zones, each individually well mixed, and between which suspended sediment can diffuse. Zone 1 contains the pond volume that is involved in short-circuiting and in which flow-dependent settling occurs, while Zone 2 contains the remaining pond volume in which quiescent settling occurs. Inflow to, and outflow from, the pond take place only in Zone 1. The conservation of sediment in the zones is described by the following two equations, which equate the rate of change of sediment mass to the sum of sediment fluxes:

$$\text{Zone 1} \quad \frac{d(V_1C_1)}{dt} = Q_iC_i - Q_oC_1 - \varepsilon(C_1 - C_2) - A_1U_1C_1 \tag{2}$$

$$\text{Zone 2} \quad \frac{d(V_2C_2)}{dt} = -\varepsilon(C_2 - C_1) - A_2U_2C_2 \tag{3}$$

where V is volume, C is suspended sediment concentration, Q is flow rate, ε is the inter-zone diffusion rate, A is surface area and U is settling velocity. Subscripts 1, 2, i and o refer, respectively, to Zone 1, Zone 2, inflow and outflow. In equation (3) the fluxes are inter-zone diffusion and settling, whilst equation (2) also contains inflow and outflow fluxes. Equations (2) and (3) were solved using a similar finite difference form as for the flow model to give the temporal variation of C_1 and C_2, for a specified distribution of sediment in the inflow. Suitable values of ε and U were selected and it was assumed that volumes, flows and surface areas were known from the flow model.

Simulations

The models were used to simulate sediment removal in an urban stormwater pond typical of those constructed in Scotland during the 1990s. To ensure that the simulated pond volume was realistically matched to inflow events, the simulations were based on Linburn Pond, located in the Dunfermline Eastern Expansion (DEX) development in eastern Scotland (56°4'N, 3°24'W). The pond was modelled with a single 90° weir, with the weir crest 3 m above the base of the pond. Simulations focused on the 24 h

duration inflow event, which is representative of the hydrological conditions in eastern Scotland. Inflow hydrographs were triangular and symmetrical with the peak inflow occurring after 12 h. The analysis of a 30-year daily rainfall record for Tullyallan (23 km distant), combined with a simple rainfall–runoff model (Morgan, 2007) showed that the peak inflow of the 1 in 25 year, the 1 in 2 year and the Q90 events were 250, 125 and 28.7 L s^{-1}, respectively.

In all simulations, the inflow sedigraph consisted of a symmetrical triangular distribution with a peak concentration of 100 mg L^{-1}. For the 1 in 2 year event, the duration of the sedigraph was 9.6 h, corresponding to the time taken for 12–15 mm of the runoff generating rainfall to have occurred. For the Q90 event, however, the runoff generating rainfall was less than 12–15 mm, so it was assumed that the sedigraph duration was the same as for the inflow hydrograph, i.e. 24 h. Since it is well known that the sediment in the runoff from urban developments consists of particles of various sizes, for each simulation case the model was run five times using sediment of different nominal sizes. The final sediment removal results were then calculated as a weighted average of the five individual simulations, the weighting being based on the typical particle size distribution of sediment in the inlets to urban stormwater ponds in the DEX development (Morgan, 2007).

In Zone 2 values of quiescent settling velocity were as given by Ellis *et al.* (1995). In Zone 1 flow-dependent settling followed a simplified version of the Hjulstrom curve (Hjulstrom, 1935) and varied linearly between zero (high flows) and the quiescent settling velocity (low flows) (Morgan, 2007). To undertake the sediment simulations the inter-zone diffusion rate (ε) and the ratio of the volumes of Zone 1 to Zone 2 needed to be calibrated. From trial simulations undertaken to find a suitable combination of these constants, ε was set at 0.01 m^3 s^{-1} and the volume ratio at 1/9. Under these conditions, the model of Linburn Pond captured about 30% of incoming sediment, which is typical of its known performance (Morgan, 2007). All the simulations were undertaken using a time step of 0.24 h, based on the results of a time-step sensitivity analysis (Morgan, 2007).

The first set of simulations explored sediment removal in ponds designed for flow attenuation or for water quality enhancement. There is no widely accepted definition of the level of flow attenuation that a stormwater pond should provide. Here the criterion adopted was that the former pond should be sized to reduce the peak outflow to half the peak inflow for the 1 in 25 year event. The basis of this criterion was early studies showing that urbanisation can increase peak flows to between two and five times those of the pre-developed catchment (e.g. Savini & Kammerer, 1961). Simulations with the flow model showed that a pond of radius 75 m and permanent pool volume of 53 014 m^3 was required to achieve this. The pond designed for water quality enhancement was sized so that the permanent pool volume was equal to one treatment volume (Vt), following the recommendations of McLean *et al.* (2005). For Linburn Pond, Vt was calculated to be 2550 m^3 for 15 mm rainfall depth, so that the radius of a stormwater pond designed in this way would be 16.45 m ($(2550/(3 \times \pi))^{0.5}$), recalling that the outlet weir crest is located 3 m above the pond base. Simulations were then carried out that considered sediment removal in both these ponds, focusing on more frequent events (1 in 2 year and Q90) because it is believed that such events may be more influential than rare flow events for pond water quality (McLean *et al.*, 2005).

As well as simulating the single Q90 event for both pond designs, some multiple Q90 event scenarios were also undertaken for both pond designs. Two scenarios were considered. Firstly, a second identical inflow event occurring immediately after the first event, but with no sediment in the second inflow, and secondly, a second identical inflow event occurring immediately after the first event with sediment in the second inflow. The former scenario is more realistic since there is limited opportunity for sediment accumulation on urban surfaces between storm events, whilst the latter scenario represents "worst case" conditions.

RESULTS AND DISCUSSION

A typical model output (Fig. 1) shows flow rates and sediment concentrations for the inflow and outflow of a pond designed for water quality enhancement during a single 1 in 2 year event. The flow attenuation is very poor for this pond design, because the pond's surface area is very small and because, as in all simulations, the water level in the pond was at the weir crest at the start of the simulation. The sediment concentration in the outflow reflects the two-zone model structure. The main sediment peak is controlled by passage through Zone 1, whilst the small increase in sediment concentration after 20 h is due to movement of sediment by the slower process of diffusion from Zone 2.

Results for flow attenuation and sediment removal for ponds designed for flow attenuation or water quality enhancement are shown in Table 1. In both storm events the pond designed for flow attenuation reduced the peak outflow by over 50%, meeting the requirement that the peak outflow should be at most half the peak inflow, in contrast to the very poor flow attenuation in the pond designed for water quality. The

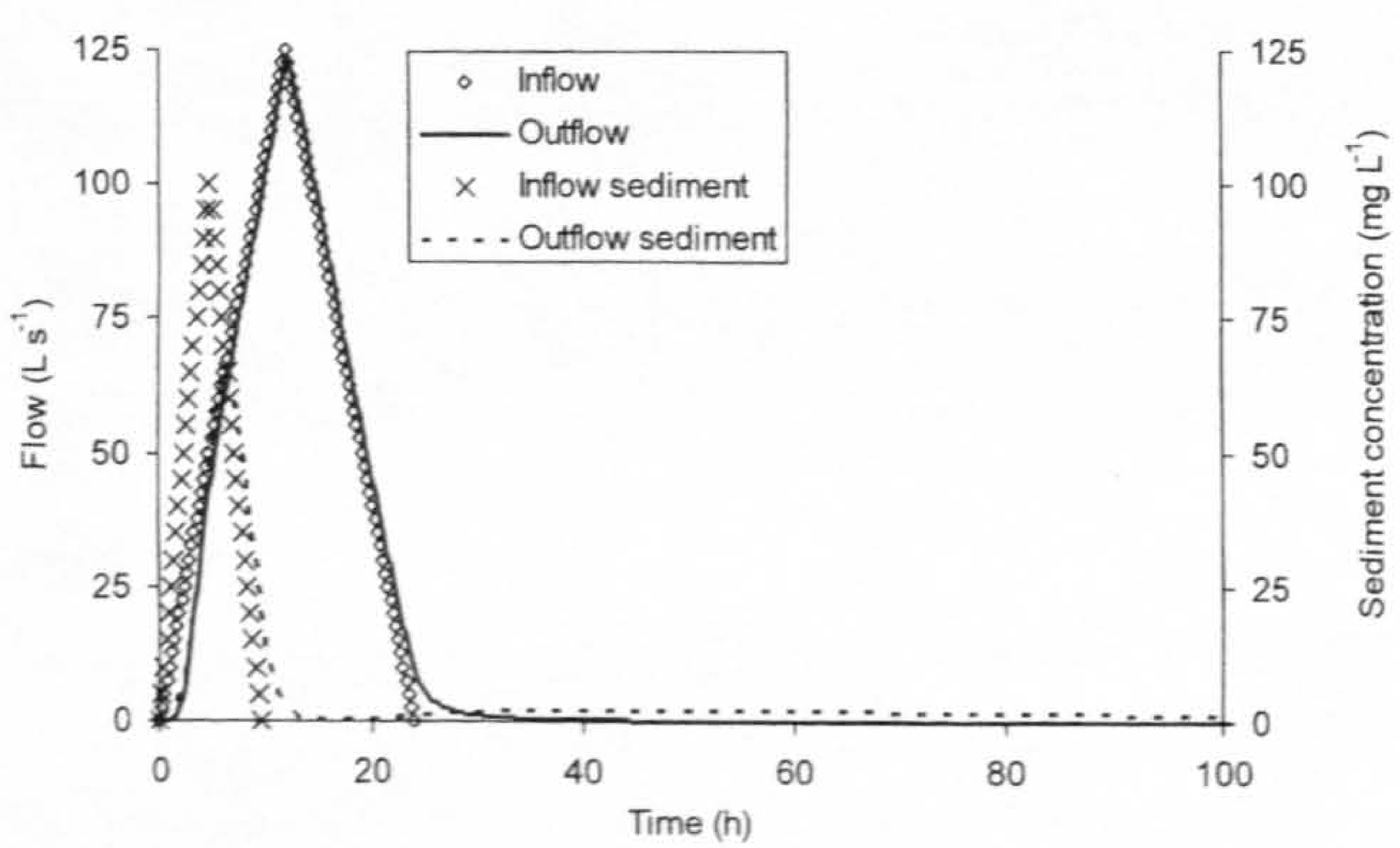

Fig. 1 Flow and sediment concentrations in the inflow and outflow of a pond designed for water quality attenuation during a 1 in 2 year storm event. The sediment concentrations shown are for the finest (<0.001 mm diameter) and dominant particle size (79% by mass).

Table 1 Peak flow reduction and sediment removal in ponds designed for flow attenuation and water quality enhancement for the Q90 and 1 in 2 year storm events.

Pond design	Storm event	Peak inflow ($L\ s^{-1}$)	Peak inflow reduction (%)	Inflow sediment mass settled (%)
Flow attenuation	Q90	28.7	94	98
Flow attenuation	1 in 2 year	125	69	78
Water quality	Q90	28.7	4.0	48
Water quality	1 in 2 year	125	2.0	21

pond designed for flow attenuation had higher sediment removal efficiencies than the pond designed for water quality enhancement in both storm events.

Most design guidance for urban stormwater ponds focuses on attenuation of individual storm events and assumes that the pond is drained down to the permanent pool prior to a storm event. However, in temperate climates, such as the UK, storms may occur in quick succession, often with very short intervening dry antecedent periods. Consequently, if there has been a recent storm event and little storage volume remains in the pond, the attenuation of subsequent rainfall events is expected to be significantly reduced. The sediment removal and inflow peak reduction when single and multiple Q90 storm events and sediment inputs were routed through ponds designed for flow attenuation or water quality enhancement is shown in Fig. 2.

In all cases, sediment removal was higher in the pond designed for flow attenuation than the pond designed for water quality enhancement for the same storm event sequence (Fig. 2(a)). Surprisingly sediment removal efficiency remained almost unchanged when multiple rather than single storm events were routed through ponds of both designs. The reason suggested to explain this is the large size of the permanent

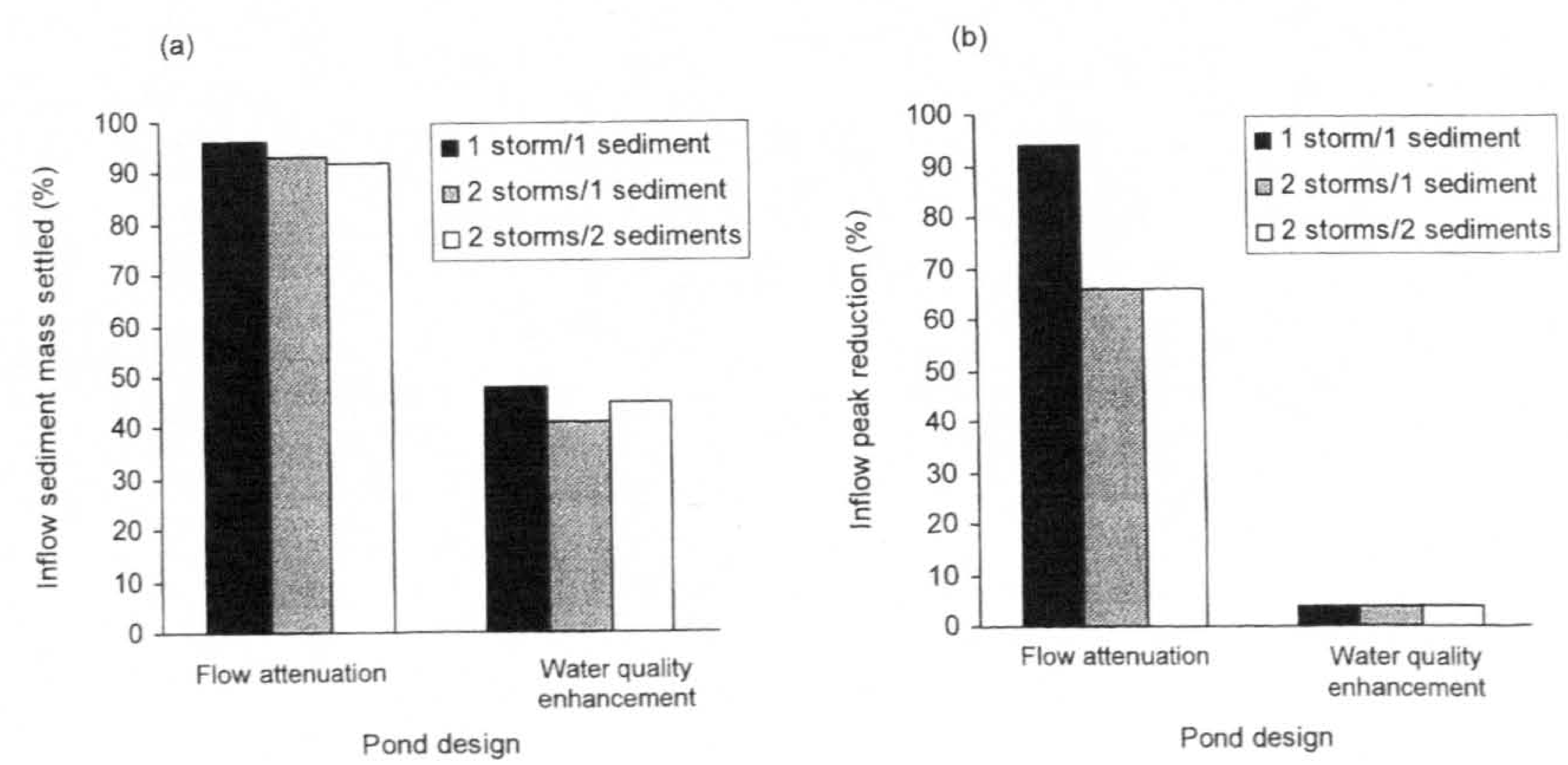

Fig. 2 (a) Sediment removal and (b) inflow peak reduction in ponds designed for flow attenuation and water quality enhancement for single and multiple Q90 storm events.

pools compared to the individual storm event volume (1240 m^3), which facilitated sediment settling before the onset of the second storm event. When the simulations were repeated for a generic infiltration pond with no permanent pool of water and sized to meet the flow attenuation criterion, sediment removal did decline from 53% in the single storm event to 39% in the two storms–one sediment input scenario. Furthermore, multiple storm events had a considerable impact on flow attenuation (Fig. 2(b)). In the pond designed for flow attenuation the peak inflow was only reduced by 69% in sequential storm events compared to 94% in a single storm event, whilst flow attenuation was very poor in the pond designed for water quality in all simulations.

Climate change is likely to produce an increase in the frequency and magnitude of rainfall events, which is expected to impact on the flow attenuation and water quality enhancement performance of urban stormwater ponds designed according to the current recommended methodology. Recent global climate model simulations have predicted increases in the frequency and intensity of heavy rainfall at northern latitudes (Ekström *et al.*, 2004), consistent with observations of changing rainfall intensity in the UK (Fowler & Kilsby, 2003). The results presented in Fig. 2 show that more frequent occurrence of small storm events will have little effect on sediment removal by urban stormwater ponds under Scottish conditions, although flow attenuation performance is very significantly impacted. However, the simulations conducted with different storm sizes showed that sediment removal decreased in both pond designs with increasing storm event magnitude from Q90 to the 1 in 2 year event (see Table 1).

CONCLUSIONS

Using a generic modelling approach it has been demonstrated that the methodology used to design urban stormwater ponds, i.e. whether they are designed for flow attenuation or for water quality enhancement, has a significant effect on pond performance. Ponds designed for flow attenuation are more successful in terms of both flow *and* pollutant attenuation (measured here as removal of suspended sediment).

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